Quantification of the Physical Impacts of Climate Change on Beach Shoreline Response

Christopher Dylan Huxley

A thesis submitted for the degree of Master of Philosophy at

The University of Queensland in February 2011

School of Engineering, Civil Engineering

Declaration by Author

This thesis is composed of my original work, and contains no material previously published or written by another person except where due reference has been made in the text. I have clearly stated the contribution by others to jointly-authored works that I have included in my thesis.

I have clearly stated the contribution of others to my thesis as a whole, including statistical assistance, survey design, data analysis, significant technical procedures, professional editorial advice, and any other original research work used or reported in my thesis. The content of my thesis is the result of work I have carried out since the commencement of my research higher degree candidature and does not include a substantial part of work that has been submitted to qualify for the award of any other degree or diploma in any university or other tertiary institution. I have clearly stated which parts of my thesis, if any, have been submitted to qualify for another award.

I acknowledge that an electronic copy of my thesis must be lodged with the University Library and, subject to the General Award Rules of The University of Queensland, immediately made available for research and study in accordance with the *Copyright Act 1968*.

I acknowledge that copyright of all material contained in my thesis resides with the copyright holder(s) of that material.

Statement of Contributions to Jointly Authored Works Contained in the Thesis

No jointly-authored works.

Statement of Contributions by Others to the Thesis as a Whole

No contributions by others

Statement of Parts of the Thesis Submitted to Qualify for the Award of Another Degree

None

Published Works by the Author Incorporated into the Thesis

None

Additional Published Works by the Author Relevant to the Thesis but not Forming Part of it

None

Acknowledgements

I would like to acknowledge the NSW Department of Environment and Climate Change and BMT WBM for sponsoring my research study. I would also like to thank my research supervisors, Professor Peter Nielsen and Doctor Ian Teakle and fellow colleague Dean Patterson for their assistance throughout the study.

Abstract

Assessing the possible impacts of future climate change is a major issue currently requiring attention by governments worldwide. By assessing the likely impacts of climate change for the full range of environmental and regional sectors, governments will be able to prioritise adaptive strategies to manage the climate change impacts in an economically responsible way.

This study represents a portion of a NSW state government funded study assessing the possible impacts of climate change to the NSW coastline. As part of this study, a modelling approach capable of assessing the impact of climate change to the existing shoreline has been developed.

The developed modelling approach, based on existing coastal engineering theory, has been designed specifically to assess the impact of climate change on shoreline response for an open coast beach. The time stepping model further develops the Miller and Dean (2004) cross-shore model using a geometric representation of the cross-shore profile. The geometric approach has been applied using Bruun Rule style conservation of mass principles.

To assess the possible shoreline response to the combined impact of climate change driven sea level rise and variations in wave climate, developed cross-shore and longshore models have been dynamically linked. Combining of the modelling programs has enabled an efficient way to assess the likely impacts of climate change on shoreline response.

Specifically, the assessment of the likely impacts of climate change variations predicted by McInnes *et al.* (2007) on Wooli Wooli Beach, summarised in Table 1-1, has been undertaken using this developed modelling approach. A summary of the most extreme model scenario results for Wooli Wooli Beach is shown in Table 1-2.

Table 1-1 Wooli Wooli Climate Change Forcings

Assessment Scenario		Climate Change Parameter		2000	2030	2070	2100 [b]
Existing Wave Climate (Exg)		Sea Level (m)[a]		0	0	0	0
		Storm Wave Height (m) [c]	NE	0	0	0	0
			E	0	0	0	0
			SE	0	0	0	0
			S	0	0	0	0
		Swell Direction (degrees)		0	0	0	0
		Swell Wave Height (m)		0	0	0	0
Climate Change Scenarios	**SLR** (Sea Level Rise Only)	Sea Level (m)[a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	0	0	0
			E	0	0	0	0
			SE	0	0	0	0
			S	0	0	0	0
		Swell Direction (degrees)		0	0	0	0
		Swell Wave Height (m)		0	0	0	0
	Global Climate Model CCM2	Sea Level (m) [a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	-0.10	+0.20	+0.43
			E	0	-0.10	+0.10	+0.25
			SE	0	+0.30	-0.10	-0.40
			S	0	+0.10	-0.10	-0.25
		Swell Direction (degrees)		0	-3.10	-3.30	-3.45
		Swell Wave Height (m)		0	0	-0.10	-0.18
	Global Climate Model CCM3	Sea Level (m) [a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	+0.20	+0.40	+0.55
			E	0	+0.10	0	-0.08
			SE	0	-0.10	0	+0.08
			S	0	-0.10	-0.10	-0.1
		Swell Direction (degrees)		0	+0.60	-1.30	-2.73
		Swell Wave Height (m)		0	0	+0.10	+0.18

[a] Sea Level Rise = NSW DECCW sea level rise benchmark (DECCW, 2009)
[b] Value extrapolated from 2030 and 2070 CSIRO prediction (McInnes *et al.*, 2007)
[c] Applied to events exhibiting wave heights greater than 3m Hsig

Table 1-2 Wooli Wooli Climate Change Maximum Shoreline Recession Results

Result	*Scenario*	Planning Horizon		
		2030	*2070*	*2100*
Maximum Shoreline Change (MLSP)	EXG	-36.1	-36.1	-36.1
	SLR Only	-47.0	-62.2	-73.4
	CCM2	**-53.4**	-63.8	-79.5
	CCM3	-47.6	**-67.1**	**-82.5**
Maximum Climate Change Related Shoreline Recession (Climate Change Scenario minus Existing Case Scenario Result)	SLR Only	-10.9	-26.1	-37.3
	CCM2	**-17.3**	-27.7	-43.4
	CCM3	-11.5	**-31.0**	**-46.4**

Summarising the findings specific to Wooli Wooli Beach, using the developed assessment approach, accounting for the combined impact of climate change on cross shore and longshore sediment transport processes, the results indicate there is a significant non-uniform alongshore response to the climate change forcing. The results show that greatest shoreline recession is likely to occur along the southern section of Wooli Wooli beach, adjacent to the northern training wall of the Wooli Wooli River. North of this location the shoreline recession results are reduced, with the northern section of Wooli Wooli beach exhibiting the smallest recession.

Alongshore, for the three assessed climate change scenarios, at the southern end of Wooli Wooli beach where the predicted erosion is of the greatest magnitude between 73m and 83m of shoreline recession is predicted.

Comparing the various scenario results, it is evident that the projected increases in sea level dominate the shoreline response at Wooli Wooli. Projected changes in wave climate are predicted to have an effect of the future shoreline evolution, though the magnitude of change is comparably less than that resulting from sea level rise in isolation.

In the broader context of shoreline response to climate change, the Wooli Wooli assessment results highlight some interesting trends. The results indicate that shoreline response to sea level rise is highly non-uniform for littoral drift dominated coastlines. Where the annual net longshore transport pattern dominates, such as at Wooli Wooli Beach, the modelling results indicate beach sections immediately downdrift from major headland/groyne controls are likely to experience the greatest shoreline recession.

These results highlighted the need to account for both cross shore and longshore processes during climate change assessments for littoral drift dominated coastlines, typical of northern NSW. Traditional climate change assessments using the Bruun Rule, represent only a cross shore assessment, and do not account for the joint interaction of the shoreline evolution to combined cross shore/longshore processes. Using the developed time-stepping model approach outlined in this paper, more detailed shoreline response assessments are now able to be conducted, accounting for these complex sediment transport processes.

Keywords
Climate Change Impact, Sediment Transport, Shoreline Response

Australian and New Zealand Standard Research Classifications (ANZSRC)

960303 - CLIMATE AND CLIMATE CHANGE - Climate Change Modelling 60%

960307 - EFFECTS OF CLIMATE CHANGE AND VARIABILITY ON AUSTRALIA (excl. Social Impacts) - 40%

CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

BMT WBM	Company name of organisation sponsoring this research study
^{o}C	Degrees Centigrade
CCM2	CSIRO Regional Climate Models: CCM2
CCM3	CSIRO Regional Climate Models: CCM3
CH_4	Methane
CO_2	Carbon Dioxide
CSIRO	Australian Commonwealth Institute and Research Organisation
DCC	Federal Department of Climate Change
DECCW	NSW Department of Environment and Climate Change and Water
DEM	Digital Elevation Model
DIPNR	NSW Department of Infrastructure Planning and Natural Resources
E	East
Exg	Existing climate
Fortran	Computer programming software
H_b	Breaking Wave Height
H_{rms}	Root Mean Squared Wave Height
H_{sig}	Significant Wave Height
IPCC	Intergovernmental Panel on Climate Change
kg	Kilograms
LSMOD	Long-shore sediment transport model developed during this study
m	Metres
m^3/m	Metres cubed volume of sand per metre beach length
Matlab	Computer programming software
MLSP	Most Landward Shoreline Position
mm	Millimetre

NE	North East
NSW	New South Wales
ppm	Parts per million
S	South
s	Seconds
SE	South East
SLR	Sea Level Rise
SOI	Southern Oscillation Index
S-SE	South South East
SWAN	Wave spectra model developed by DELTF Hydraulics
T_p	Peak wave period
USA	United States of America
XSMOD	Cross shore sediment transport model developed during the thesis study

Table 1-3 Equation Parameter Summary

Equation	Parameter	Description
1	$$Q_l = K\left[\frac{\rho\sqrt{g}}{16\gamma^{\frac{1}{2}}(\rho_s-\rho)(1-n)}\right]H_b^{\frac{5}{2}}\sin(2\alpha_b)$$	
	K	Dimensionless Coefficient (based on significant wave height calculations approximately = 0.39)
	ρ_s	Sediment Density (for quartz density sand approximately = 2,650kg/m^3)
	ρ	Water Density (for salt water = 1,025 kg/m^3)
	g	Acceleration Due to Gravity (approximately = 9.8m/s^2)
	N	Sand Porosity (approximately = 0.4)
	γ	Breaker Index (approximately = 0.78)
	α_b	Shore Normal Wave Angle (degrees)
	H_b	Significant Wave Height at Breaker Depth (m)
2	$$Q = 6.4\times10^4 H_{sb}^2 T_p^{1.5} m_b^{0.75} D_{50}^{-0.25}\sin^{0.6}(2\alpha_b)$$	
	H_b	Wave Height at Breaker Depth (m)
	T_p	Peak Wave Period (s)
	m_b	Nearshore bed slope (m/m)
	D_{50}	Median Grain Size (m)
	α	Shore Normal Wave Angle (degrees)
3	$$y = Ax^{2/3}$$	
	y	Bed Elevation (m)
	A	Sediment Scale Parameter (m$^{1/3}$)
	x	Distance Offshore (m)

Equation	Parameter	Description
4, 25	$$y=\frac{Ax^{2/3}-\overline{\eta}_b-\mathrm{K}h_b}{(1-\mathrm{K})}$$	
	y	Bed Elevation (m)
	A	Sediment Scale Parameter ($m^{1/3}$)
	x	Distance Offshore (m)
	$\overline{\eta}$	Mean water level at breaking (m)
	γ	Breaker index
	h_b	Water depth at breaking (m)
	K	See Equation 5
5	$$\mathrm{K}=\frac{3\gamma^2/8}{1+3\gamma^2/8}$$	
	γ	Breaker index
6	$$(7.6/H_0)y=0.47\left[(7.6/H_0)^{1.28}(w/0.0268)^{0.56}x+18\right]^{0.5}-2.00$$	
	H_0	Significant Deep Water Wave Height (m)
	w	Sediment Fall Velocity (m/s)
	y	Elevation (m)
	X	Distance Offshore (m)
	γ	Breaker index
7	$$s=l\frac{a}{h}$$	
	s	Horizontal Shoreline Recession (m)
	a	Water Level Increase (m)
	l	Width of Active Profile (m) Defined by the upper limit of the active profile and the offshore depth of closure.
	h	Height of the active profile (m) = $h_{\text{depth of closure}} + h_{\text{berm}}$
8	$$R(t)=R_\infty(1-e^{-\frac{t}{T_s}})$$	
	R(t)	Shoreline position at time (t) (m)
	$R\infty$	Maximum shoreline position (m)
	T_s	Erosion time scale (hours)

Equation	Parameter	Description
15,22	$$y^{n+1} = \frac{y^n + A\left[y_{eq}^{n+1} + y_{eq}^n - y^n\right]}{1 + \frac{k\Delta t}{2}}$$	
	y^{n+1}	Shoreline Position- Future Timestep (m)
	y^n	Shoreline Position- Current Timestep (m)
	y_{eq}^{n-1}	Maximum "Equilibrium" Shoreline Position- Previous Timestep (m)
	y_{eq}^{n}	Maximum "Equilibrium" Shoreline Position- Current Timestep (m)
	k	Rate Coefficient (sec^{-1}) (Note: separate coefficients for erosion k_e and accretion k_a)
	Δt	Timestep (s)
	A	Sediment Scale Parameter (dimensionless)
16,21,28	$$y_{eq}^n(t) = y_0 + W*(t)\left(\frac{0.106H_b(t) + S}{B + 2.0H_b}\right)$$	
	y_{eq}^{n}	Maximum "Equilibrium" Shoreline Position- Current Timestep (m)
	y_0	Empirical Shoreline Adjustment Coefficient (m)
	$W*(t)$	Width of Active Surf Zone (m)
	$H_b(t)$	Breaking Significant Wave Height (m)
	S	Storm Surge Level (m)
	B	Berm Height (m)
17	$$\alpha_s = a\tan\left[\frac{(xc_{n-1} - xc_n)}{dx}\right]$$	
	xc	Cell centre length (m)
18	$$dy = \frac{dt(Q_{n-1} - Q_n)}{dx\left(D_b + D_c\right)}$$	
	dt	Timestep (s)
	dx	Cell Width (m)
	Q	Sediment transport (m^3/s)
	D_b	Berm Height (m)
	D_c	Depth of Closure (m)

Equation	*Parameter*	Description
23,24,26,27	$y = B$ $y = y_{d0} - m_0(x - \Delta x)$ $y = \frac{-Ax^{\frac{2}{3}} - n_b - Kh_b}{1-K}$ $y = \frac{1}{-m_1}x + b_1$	
	For parameter description refer to Figure 4-6	

1 AIM OF STUDY

1.1 Background

Climate change is rapidly becoming one of the pre-eminent issues for governments worldwide. Lomborg (2007) summarises,

> *"The European Union calls it "one of the most threatening issues that we are facing today." Former Prime Minister Tony Blair of the United Kingdom sees it as " the single most important issue." German chancellor Angela Merkel has vowed to make climate change the top priority within both the G8 and the European Union in 2007…(Within the United States of America) several coalitions of states have set up regional climate initiatives."*

In the Australian perspective, the Department of Climate Change (DCC) was established on the 3rd of December 2007, formerly a part of Department of the Environment and Water Resources. The role of the DCC focuses on ensuring Australia meets its responsibilities in facing the global challenge of climate change. This includes a comprehensive approach to:

- Reduce greenhouse gas emissions in Australia in the short and long term;
- Working with the international community to develop a global response that is effective and fair; and
- Preparation for the inevitable impacts of climate change.

(Australian Government Department of Climate Change, 15/2/2008)

On a state level, in New South Wales (NSW), the department responsible for the management of the environment, natural resources, natural and cultural heritage and climate change impacts rests with the NSW Department of Environment and Climate Change and Water (DECCW) (DECCW, 30/1/2008).

The issues relating to climate change are vast. Some of the projected impacts of climate change stated by the Intergovernmental Panel on Climate Change (IPCC), the premier authority in climate change science include:

- Global average temperatures are likely to increase;
- Global mean sea levels are likely to rise;
- Increased acidification of the ocean is likely to occur;
- The ocean thermocline is likely to become a stronger barrier against mixing;
- Snow cover is predicted to contract;
- Sea ice cover is predicted to shrink;

- Extreme meteorological events such as heat waves and heavy precipitation events are likely to become more frequent;
- Future tropical cyclones are likely to become more intense;
- Extratropical storm tracks are projected to move poleward; and
- Increases in precipitation are likely to occur in high latitudes, whilst decreases in precipitation are likely in most subtropical land areas.

(IPCC, 2007)

These climate change impacts have the potential to cause major shifts/changes to existing environmental, social and economic systems. Voice *et al.* (2006) provides a thorough assessment of the vulnerability of Australia's coastal zone to climate change forcings. Voice *et al.* (2006) splits the coastal zone into the following categories:

- Beaches and dune coasts;
- Estuaries;
- Mangroves;
- Seagrasses;
- Corals and coral reefs;
- Coastal Infrastructure and water resources;
- Fisheries and aquaculture; and
- Selected other coastal activities.

Without detailed studies predicting the possible impacts of climate change for the areas of interest listed above, the DCC and the DECCW will not adequately be able to meet one of their main objectives, to "prepare for the inevitable impacts of climate change" (Australian Government Department of Climate Change, 15/2/2008) in the coastal zone.

Studies of coastal vulnerability of beaches and sandy coasts in Australia can be categorised into three types of study in terms of their detail and scale:

1. National studies relating to climate change which contain references to vulnerability of sandy coastlines;
2. State based strategic planning documents identifying coastal areas at risk; and
3. Local case studies focusing on specific locations commonly academic/scientific based studies and local government consultancy studies driven by a needs based approach for assessing risks in local government areas. (Voice *et al.*, 2006)

State and national studies worthy of mention include the:

- National Coastal Vulnerability study (Department of Environment, State and Territories, 1996);
- Victorian Coastal Vulnerability Study (Port of Melbourne Authority, 1992);
- Tasmanian Coastal Vulnerability Study (Sharples, 2004);

With the exception of the Victorian study these studies represent a first pass assessment identifying vulnerable coastal zones, though the national study has been criticised for inconsistencies in its methodology (Kay and Waterman 1993). The Victorian study was more detailed, including Bruun Rule and recession modelling for 21 selected beaches within Victoria. Due to the date of this study, however, the climate change assumptions adopted are somewhat outdated.

1.2 DECCW Climate Change Study

The overarching objective of the DECCW study is to assess the environmental and economic impact of potential coastal erosion, coastal inundation and degradation of estuaries due to climate change in coastal NSW.

It is recognised that the exact response of any beach or estuary to a given set of environmental forcing functions (eg. storm waves, storm surge, rainfall events, tidal range, and salinity regime) will depend on the site-specific geomorphic features. However, a comprehensive study covering the entire 1000km long NSW coastline is not possible due to budget and time constraints. Fortunately, the NSW coastline has traditionally been considered as two distinct units based on littoral drift characteristics. These littoral drift units govern coastal processes such as storm erosion, sand bar migration and estuarine processes such as entrance condition, ebb and flood tidal flows. The southern and central NSW coastlines are typically low-littoral drift coastlines (ie. swash dominated) whereas the northern NSW coastline is typically a high-littoral drift coastline (ie. drift dominated) with measured littoral drift rates up to 500,000 m^3/yr. Similarly, NSW estuaries generally reflect the differences in rainfall, geology and wave climate along the coast and may be categorised into three main types; drowned river valleys, wave dominated barrier estuaries and intermittently closed and open lakes and lagoons (DNR, 2007).

Two representative sites, one each from the NSW north and south coasts, have been selected as case studies. The Wooli Wooli River system is a barrier estuary on the NSW north coast and the Clyde River/Batemans Bay system is a drowned river valley on the NSW south coast. It is envisaged that the results obtained for these two selected study areas will characterise the potential climate change impacts likely in these two geographically diverse systems. The relative impacts associated with various climate change scenarios will be qualitatively and, to some extent, quantitatively indicative of impacts likely to be observed at other locations along the NSW coastline (DNR, 2007). By focussing on only two locations a more detailed assessment predicting the possible shoreline evolution resulting from climate change can be undertaken, when compared with the previous state-wide studies mentioned above. To achieve the objectives of the DECCW study, the project has been has been split into three separate parts.

1.2.1 Stage 1 Assessment

The first component of the study (Stage 1), undertaken by the CSIRO (McInnes *et al.*, 2007), used modelling techniques to predict changes in offshore weather patterns for the east coast of Australia. These predictions extend from the present to 2030 and 2070. The McInnes *et al.* (2007) assessment was completed using predicted CO_2 concentrations, based on IPCC predictions, to drive two different climate models. The two models were selected as they represented the two models available to the CSIRO which produced the highest and lowest response to the climate change forcings. Using these two models provides the widest realistic range of change resulting from increases in CO_2. Based on the output from the meteorological models, changes to forcing parameters affecting coastal erosion and estuarine processes were derived. This included changes in:

- Average wave height, period and direction;
- Storm frequency and intensity;
- Storm surge occurrence and magnitude;
- Climate change driven sea level rise;
- Rainfall frequency, intensity and volume; and
- Solar radiation.

1.2.2 Stage 2 Assessment

This masters research project represents a sub-component of Stage 2 of the DECCW study. Stage 2 of the DECCW project has used the predicted variation in the above parameters to drive various numerical models. Stage 2 aimed to predict future changes in the following coastal and estuarine features for 2030 and 2070 at both Batemans Bay and Wooli Wooli. The defined objectives for Stage 2 are:

1. Determination of changes in shoreline position due to climate change induced long term trends such as sea level rise, variations in net littoral sediment transport and variations in short term shoreline fluctuations due to storm erosion.

2. Determination of changes in mean estuarine circulation, mixing and flushing times due to climate change driven variations in mean sea level and other environmental variables.

1.2.3 Stage 3 Assessment

Using the physical predictions identified as part of Stage 2 of the study, Stage 3 of the DECCW study will identify the economic impact of these climate change driven changes in the coastal zone.

Overall, the complete study will aid the DECCW to investigate effective adaptation strategies to inform government policy managing the impacts of climate change.

1.3 Research Masters Project

This masters research project represents a sub-component of Stage 2 of the DECCW study. Only the coastal erosion component specific to the Wooli Wooli case study location will be addressed during this assessment. As such, this research masters project aims to assess the possible response of the shoreline at Wooli Wooli due to changed weather patterns and sea level rise driven by climate change. It is expected that the methods adopted in this study will provide a benchmark for future studies of this nature for other locations in NSW (Huxley *et al.*, 2007).

The remaining component of Stage 2 addressing changes in estuarine processes and shoreline response at Batemans Bay will not be addressed as part of this study, though has been completed by BMT WBM.

2 OBJECTIVE

To develop methodologies to quantify the physical impacts, specifically the shoreline response, to climate change driven variations in wave climate and sea level.

Applying the developed methodologies, the sensitivity of Wooli Wooli beach shoreline alignment to the predicted climate change driven variations will be assessed.

3 LITERATURE REVIEW

3.1 Climate Change Predictions

It is recognised that future climate change has the potential to impact most, if not all of the earth's environmental systems.

The Intergovernmental Panel on Climate Change (IPCC), one of the worlds leading authorities regarding climate change predictions, attributes the main cause of climate change since the pre-industrial period on the use of fossil fuels and land use changes (IPCC, 2007).

3.1.1 Intergovernmental Panel on Climate Change

The IPCC is a scientific organisation established in 1988 by the World Meteorological Organisation and the United Nations Environment Programme. The main aim of the IPCC is to provide decision makers worldwide with an objective source of information about climate change.

Recent papers published by the IPCC have documented recorded data indicating that the global atmospheric concentration of carbon dioxide has increased from a pre-industrial value of approximately 280ppm to 379ppm in 2005 (IPCC, 2007). Similar trends have also been recorded for various other major greenhouse gases such as methane and nitrous oxides. Figure 3-1 shows the recorded increase in carbon dioxide since pre-industrial times.

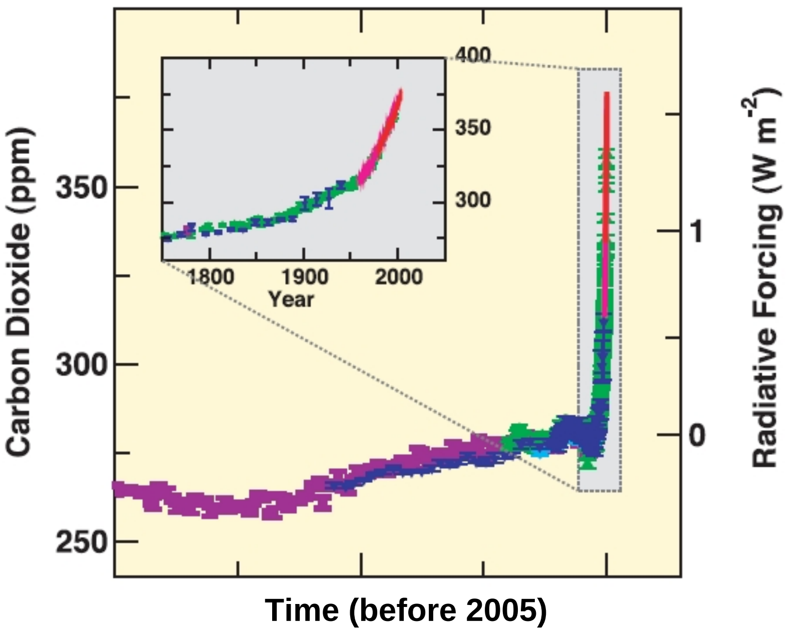


Figure 3-1 Atmospheric CO_2 Concentrations (IPCC, 2007)

The IPCC (2007) comment that these increasing trends in greenhouse gas concentration match recorded increases in global average temperature, sea level and correspond to snow cover declines in the northern hemisphere. Based on these existing trends, studies predicting the likely future temperature increases have been undertaken to inform planning decisions accommodating for the potential impacts of climate change.

Based on various future emission scenarios the IPCC have documented predicted increases in global mean temperature through till 2100. As shown in Figure 3-2, the predicted increases in global mean temperature range from 1.1°C to 6.4°C (IPCC, 2007).

Review of the IPCC results highlight significant variance in reported likely temperature increase predictions. The right hand side of Figure 3-2 shows the range in predicted temperature increase for each of the defined climate change scenarios. The variation in predictions between the listed scenarios is attributed to difficulties regarding predictions of future social behaviours. The variance for each scenario highlights the complexities associated with modelling the carbon cycle and the associated climate response accounting for the multitude of environmental sinks and feedback mechanisms.

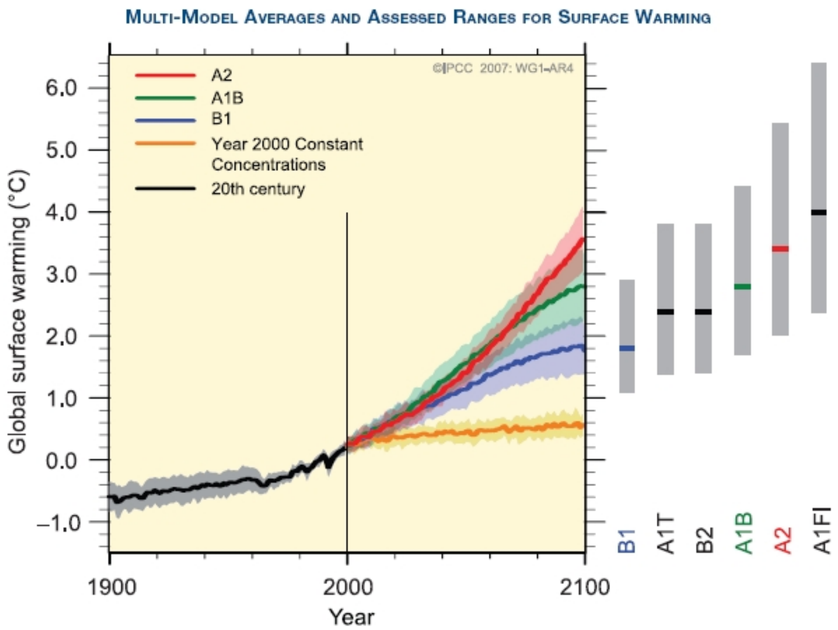


Figure 3-2 Projected Global Temperature Increases

Acknowledging the significant variance in predicted increases in global mean temperature resulting from climate change, given the availability of information, it is socially responsible to assess the potential impacts of climate change.

Specific to the coastal zone it is anticipated that increases in global mean temperature are likely to result in:

- Rises in global mean sea level; (0.18-0.59m excluding the influence of accelerated ice sheet melting in Greenland and Antarctica) (IPCC, 2007); and
- Changed offshore weather patterns resulting in varied wave climates (McInnes *et al.*, 2007).

How are these changes likely to impact on the physical aspects (beach and dune) of the coastal zone? This Masters document reviews existing theories used for shoreline response modelling.

Additionally, a new cross-shore model has been developed, to be coupled with an existing longshore model enabling the assessment of shoreline response to climate change.

3.1.2 Sea Level Rise Projections

The IPCC have provided mean global sea level rise projections for each emission scenario for the 5% and 95% confidence limits. Outlining the available global projections, Table 3-1 shows the global mean sea level projections documented in the IPCC Fourth Assessment Report. Based on the IPCC figures, including scaled up ice sheet contributions, the global sea level rise is projected to be between 18–76 cm by year 2090-2099 relative to 1980-1999 levels.

Table 3-1 IPCC Projected Global Mean Sea Level Rise (Meehl *et al.*, 2007)

		B1		B2		A1B		A1T		A2		A1FI	
Thermal expansion	m	0.10	0.24	0.12	0.28	0.13	0.32	0.12	0.30	0.14	0.35	0.17	0.41
	mm yr^{-1}	1.1	2.6	1.6	4.0	1.7	4.2	1.3	3.2	2.6	6.3	2.8	6.8
G&IC	m	0.07	0.14	0.07	0.15	0.08	0.15	0.08	0.15	0.08	0.16	0.08	0.17
	mm yr^{-1}	0.5	1.3	0.5	1.5	0.6	1.6	0.5	1.4	0.6	1.9	0.7	2.0
Greenland Ice Sheet SMB	m	0.01	0.05	0.01	0.06	0.01	0.08	0.01	0.07	0.01	0.08	0.02	0.12
	mm yr^{-1}	0.2	1.0	0.2	1.5	0.3	1.9	0.2	1.5	0.3	2.8	0.4	3.9
Antarctic Ice Sheet SMB	m	-0.10	-0.02	-0.11	-0.02	-0.12	-0.02	-0.12	-0.02	-0.12	-0.03	-0.14	-0.03
	mm yr^{-1}	-1.4	-0.3	-1.7	-0.3	-1.9	-0.4	-1.7	-0.3	-2.3	-0.4	-2.7	-0.5
Land ice sum	m	0.04	0.18	0.04	0.19	0.04	0.20	0.04	0.20	0.04	0.20	0.04	0.23
	mm yr^{-1}	0.0	1.8	-0.1	2.2	-0.2	2.5	-0.1	2.1	-0.4	3.2	-0.8	4.0
Sea level rise	m	0.18	0.38	0.20	0.43	0.21	0.48	0.20	0.45	0.23	0.51	0.26	0.59
	mm yr^{-1}	1.5	3.9	2.1	5.6	2.1	6.0	1.7	4.7	3.0	8.5	3.0	9.7
Scaled-up ice sheet discharge	m	0.00	0.09	0.00	0.11	-0.01	0.13	-0.01	0.13	-0.01	0.13	-0.01	0.17
	mm yr^{-1}	0.0	1.7	0.0	2.3	0.0	2.6	0.0	2.3	-0.1	3.2	-0.1	3.9

Regional Projections

In addition to the global increase in sea level rise discussed above, regional variation in sea level is predicted to occur along the east coast of Australia, relative to the global mean. The variation in sea level rise relative to the global mean is predominantly due to the predicted strengthening of the East Australia Current and associated thermal expansion. The CSIRO (2008) have provided estimates of the regional sea level rise contribution for 2030 and 2070 for the A1B emission scenario. Figure 3-3 shows the CSIRO regional contribution results. The CSIRO results only provide a range of ±1 standard deviation. Assuming a normal distribution this corresponds the approximately a 30% and 70% confidence limit. Although these values do not correspond exactly to the IPCC 5% and 95% values, for convenience they have been assumed to be equivalent. Likewise, although the predictions do not match the years quoted in the IPCC fourth assessment report, it has been assumed that the 2070 region contribution values can be applied to the 2100 IPCC global mean projections. This assumption results in maximum value estimates equivalent to those specified as the NSW guideline values listed in Section 3.1.3.

Assuming the CSIRO regional contribution estimate are suitable for each of the emission scenarios the minimum and maximum regional contribution to sea level rise likely to be experienced off the NSW coastline to 2100 is ranges from -0.06m to +0.14m (CSIRO, 2008). Based on these regional variation estimates and the projected global mean sea level values (Meehl *et al.*, 2007), the range of projected sea level rise values for the NSW coastline for 2100 are between 0.12m (0.18m - 0.06m) and 0.90m (0.76m + 0.14m).

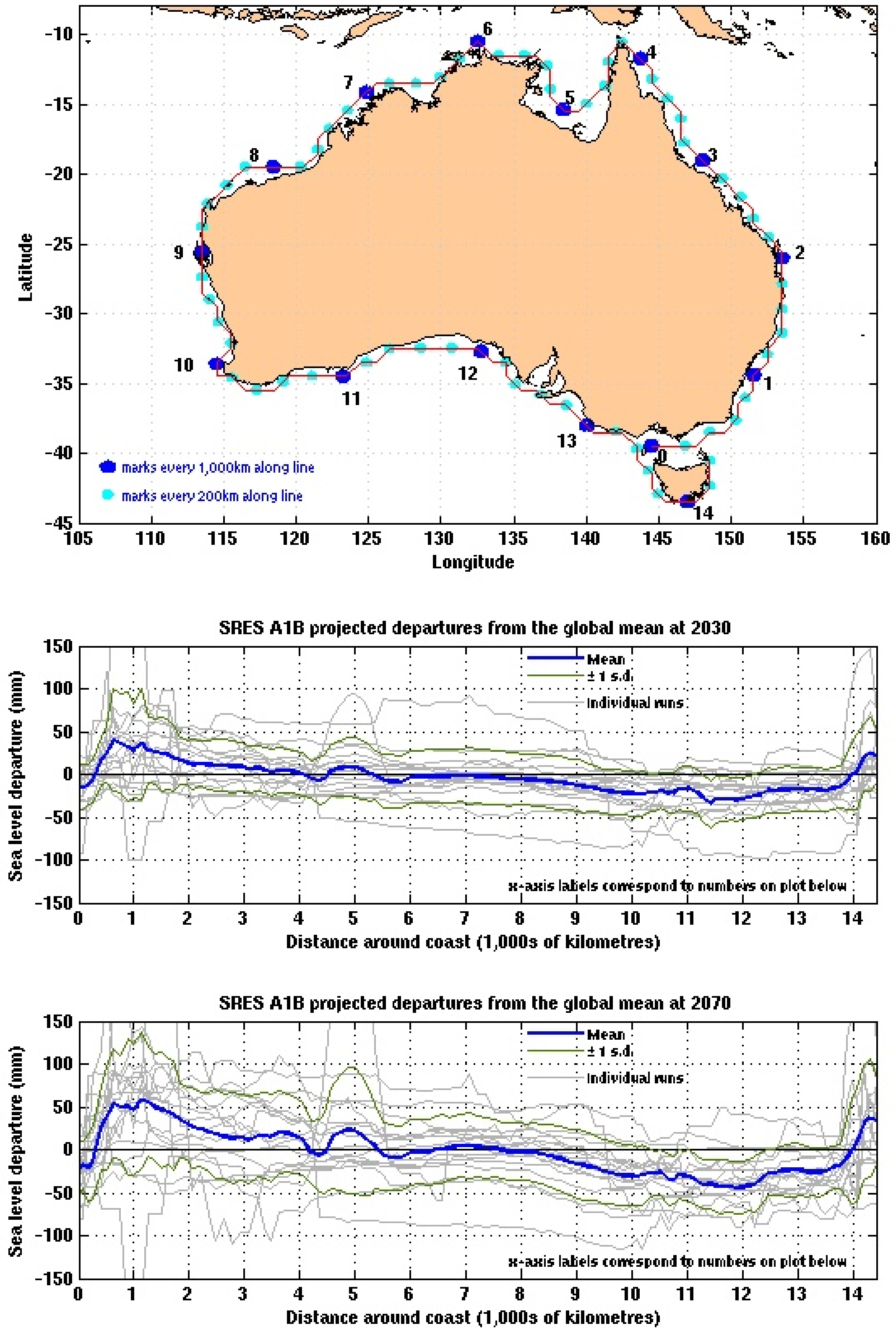


Figure 3-3 Projected Regional Sea Level Rise Contribution (CSIRO, 2008)

3.1.3 NSW Sea Level Rise Guidelines

During the course of this study the NSW DECCW released a guideline document for sea level rise to be used for planning purposes. The adopted a sea level rise planning levels are as follows:

- An increase above 1990 mean sea levels of 40cm by 2050; and
- An increase above 1990 mean sea levels of 90 cm by 2100 (DECCW, 2009).

The 2100 sea level rise planning levels match the maximum predicted sea level rise projections for NSW, stated in Section 3.1.2.

For the climate change sensitivity assessment undertaken as part of this study, Section 6.1.6.1, the adopted NSW DECCW sea level rise guideline values were used to assess the likely impact of sea level rise to the shoreline for a particular case study location.

3.1.4 Wave Climate

To fulfil the objectives of Stage 1 of the DECCW climate change project; the CSIRO undertook a study predicting the likely meteorological impacts resulting from climate change for coastal NSW. As a conservative approach the CSIRO work based its predictions on a mean global increase in temperature associated with the A2 emissions scenario. Using the results of the meteorological assessment, the CSIRO defined likely variations in wave climate resulting from climate change to the 2030 and 2070 planning horizons.

This research project is relying on the results of the CSIRO study to drive various climate change models developed as part of this masters research project predicting the physical impacts of climate change. Table 3-2 and Table 3-3 list a summary of the CSIRO results, specific for Wooli Wooli.

Table 3-2 Predicted Changes to Storm Wave Conditions – Wooli Wooli

		CCM2					CCM3				
		NE	E	SE	S	S-SE	NE	E	SE	S	S-SE
Storm Wave Mean H_{sig}	1980	3.3	4.0	3.8	3.7	3.7	3.0	3.7	3.6	3.6	3.6
	2030	3.3	3.9	4.1	3.8	3.9	3.2	3.8	3.5	3.6	3.5
	Change (m)	-0.1	-0.1	0.3	0.1	0.2	0.2	0.1	-0.1	-0.1	-0.1
	2070	3.6	4.1	3.7	3.6	3.6	3.4	3.7	3.5	3.6	3.5
	Change (m)	0.2	0.1	-0.1	-0.1	-0.1	0.4	0.0	0.0	-0.1	-0.1

Source: (McInnes *et al.*, 2007)

Table 3-3 Predicted Changes to Swell Conditions – Wooli Wooli

		CCM2			CCM3		
		Average Direction	Average Hsig	Average Ts	Direction	Average Hsig	Average Ts
Change in all Swell Waves	1980	109.2	1.3	4.5	100.7	1.2	4.5
	2030	106.1	1.2	4.5	101.3	1.2	4.6
	Change	-3.1	0.0	0.0	0.6	0.0	0.1
	2070	105.9	1.2	4.4	99.4	1.3	4.6
	Change	-3.3	-0.1	-0.1	-1.3	0.1	0.1

Source: (McInnes *et al.*, 2007)

3.2 Current Climate Change Trends

There is significant controversy regarding climate change at present. The general questions relating to climate change and its validity commonly include:

Is climate change occurring? If so, is it due to human activities or is it part of the geological cycle of the earth?

This study is not attempting to answer these questions; however, in the following sections a review of recorded trends in the earth's climate has been included. Based on the trends shown in these datasets it is apparent that the earth's climate is dynamic, and possibly warming. If these trends continue, to manage the possible impacts of these changes, governments worldwide need to be informed of the possible impacts of these changes. This study provides a means to meet these needs. As part of this study a methodology has been developed suitable for assessing the likely impact of climate change on shoreline response.

3.2.1 Greenhouse Gas Concentrations

It is believed changes in the atmospheric abundance of greenhouse gases and aerosols, in solar radiation, cloud cover and in land surface properties alter the energy balance of the climate system. These changes are expressed in terms of radiative forcing, which is used to compare how a range of human and natural factors drive warming or cooling influences on global climate (IPCC,2007). Figure 3-4 shows the relative contribution of the natural and anthropogenic factors contributing and their associated radiative forcing. As shown in Figure 3-4, greenhouse gases, particularly carbon dioxide (CO_2) and methane (CH_4) make up a large proportion of the positive radiative forcing factors. Since the industrial revolution the global concentration of these gases has dramatically increased, primarily due to the use of fossil fuels and changes in agriculture. It is likely that the increase in these greenhouse gases has resulted in the recorded increases in global mean temperature and sea level rise shown in Section 3.2.2 and 3.2.3.

Documentation of the historic changes in greenhouse gas concentrations are document in Chapter 2 of the Intergovernmental Panel on Climate Change (IPCC) Fourth Assessment Report (Forster et al., 2007). Figure 3-5, taken from the Fourth Assessment Report summary for Policy Makers (IPCC, 2007) shows the recorded concentrations in Carbon Dioxide and Methane over the past 2000 years obtained from ice core and modern data. Figure 3-5, shows the increase in carbon dioxide and methane levels from 280ppm and 715ppm during pre-industrial times to 379ppm and 1774ppm respectively in 2005. It is believed these concentration increases are largely the result of human activities.

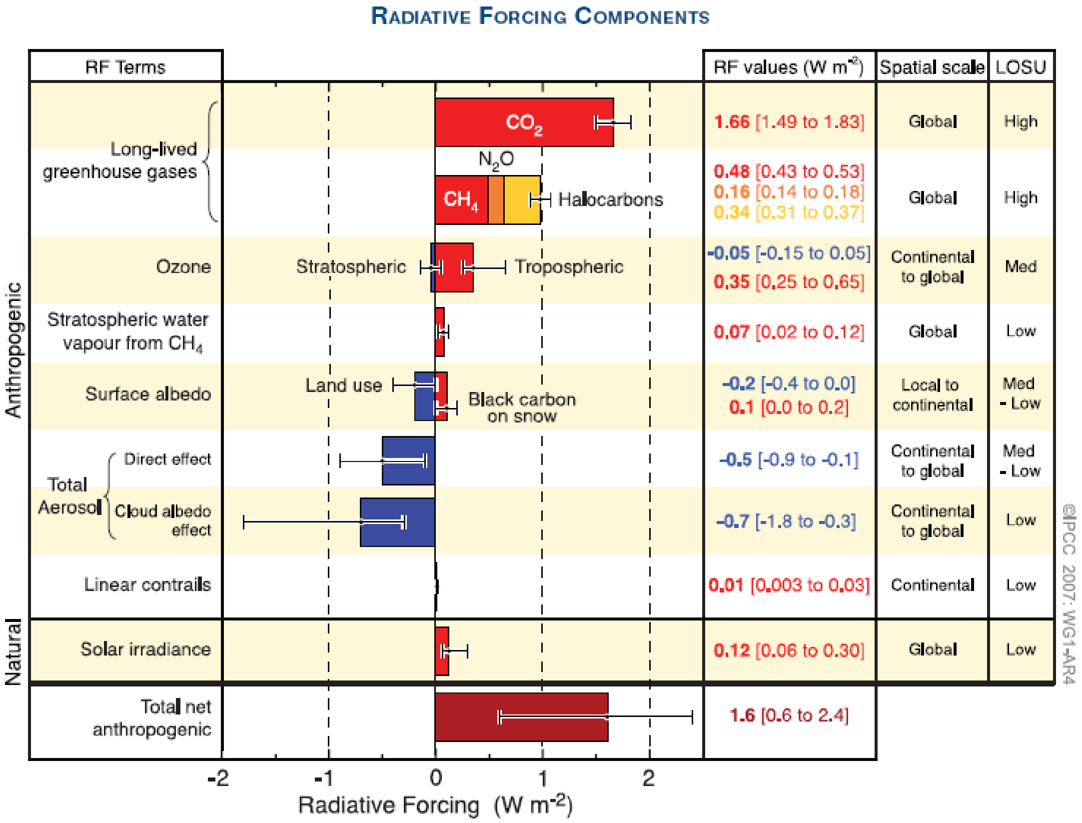


Figure 3-4 Global Atmospheric Radiative Forcings (IPCC, 2007)

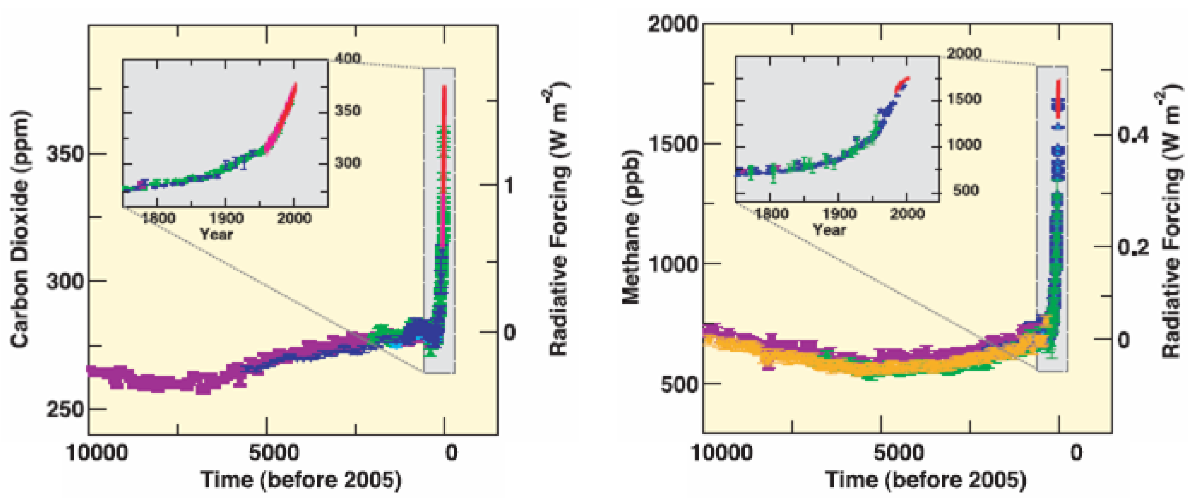


Figure 3-5 Historic Atmospheric Concentrations of Dominant Greenhouse Gasses (IPCC, 2007)

3.2.2 Global Sea Level

Various studies have reported data outlining recent changes in mean sea level. Within Chapter 5 of IPCC Fourth Assessment Report (Bindoff *et al.*, 2007), research showing the recorded changes in the mean global sea level from 1870 to 2004 are summarised. The summary figure showing these results is shown in Figure 3-6.

The historic sea level measures were obtained using worldwide tide gauge measurements and satellite altimetry. Using datum elevation correction techniques to account for the subsidence or rising of the tide stations due to tectonic movement, Church and White (2006) were able to plot the global mean sea level elevation from 1870 to 2004. Figure 3-7 shows the available tidal gauge data used by Church and White. A similar methodology was used by Holgate and Wentworth (2004). Since 1992, Satellite altimetry has been used to calculate changes in the mean global sea level; these recordings were reported by Leuliette *et al.* (2004).

Overall the sea level recordings indicate that the rate of sea level rise since the early 1900s is increasing in line with the estimates projected by the IPCC. Church and White state:

> *"… reconstruction of global mean sea level back to 1870* (results in) *a sea-level rise from January 1870 to December 2004 of 195 mm, a 20th century rate of sea-level rise of 1.7 ± 0.3 mmyr^{-1} and a significant acceleration of sea-level rise of 0.013 ± 0.006 mmyr^{-2}.This acceleration is an important confirmation of climate change simulations which show an acceleration not previously observed. If this acceleration remained constant then the 1990 to 2100 rise would range from 280 to 340 mm."*

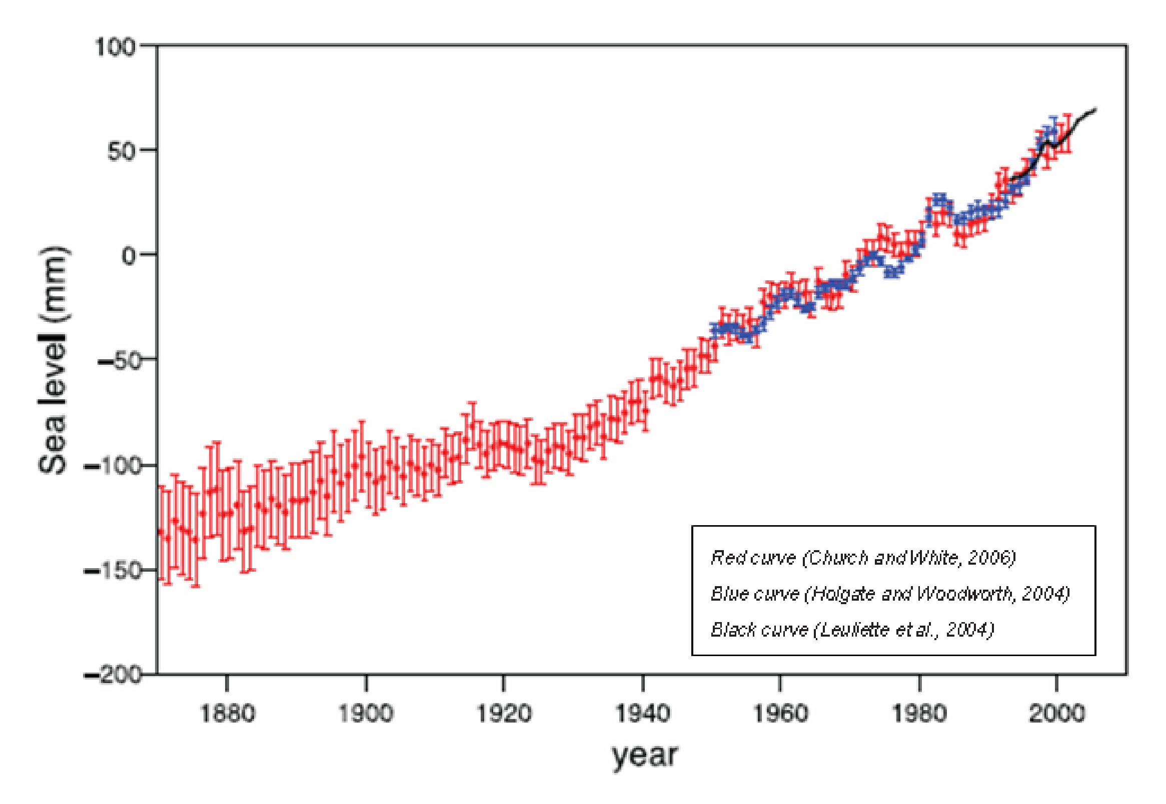


Figure 3-6 Global Mean Sea Level Change 1870 – 2004 (Bindoff *et al.*, 2007)

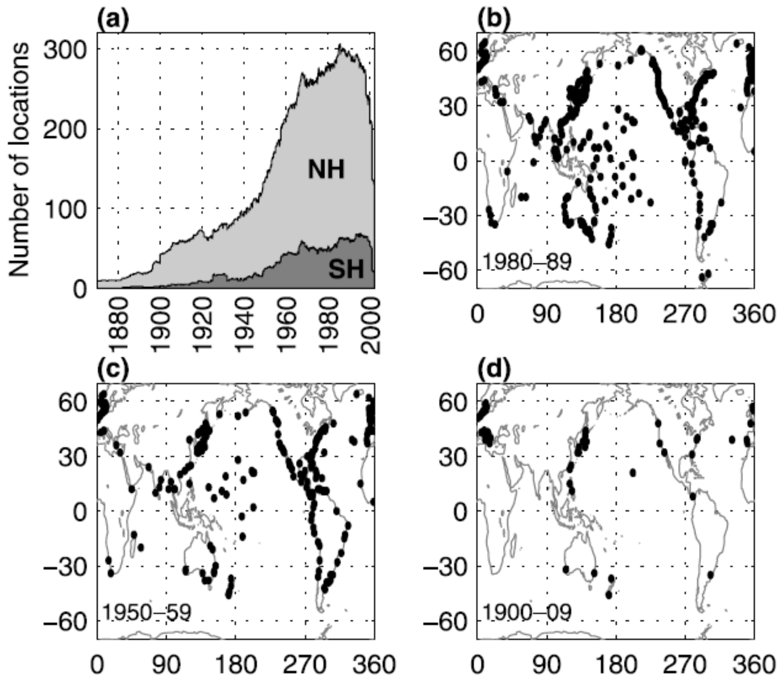


Figure 3-7 Global Mean Sea Level Records (Church and White, 2006)

3.2.3 Temperature

In addition to global sea level rise, the recorded increases in global temperature are an indication that climate change is occurring. As shown in Figure 3-8, based on the Australian Bureau of Meteorology records, the mean temperature in Australia has increase by 0.9°C from 1950 to 2008. The recorded distribution of change in maximum temperatures from 1950 to 2008 is shown in Figure 3-9. These temperature increase trends also correspond with the global trend in mean land surface temperatures documented in Chapter 3 of IPCC Fourth Assessment Report (Trenberth *et al.*, 2007), shown in Figure 3-10.

Based on the above mentioned temperature trends, and the fact that in 2007, when the Fourth Assessment Report (IPCC, 2007) was published, eleven of the previous twelve years (1995-2006) rank among the twelve warmest years in the instrumental record of global surface temperature since 1850 (IPCC, 2007), it is evident that climate change occurring.

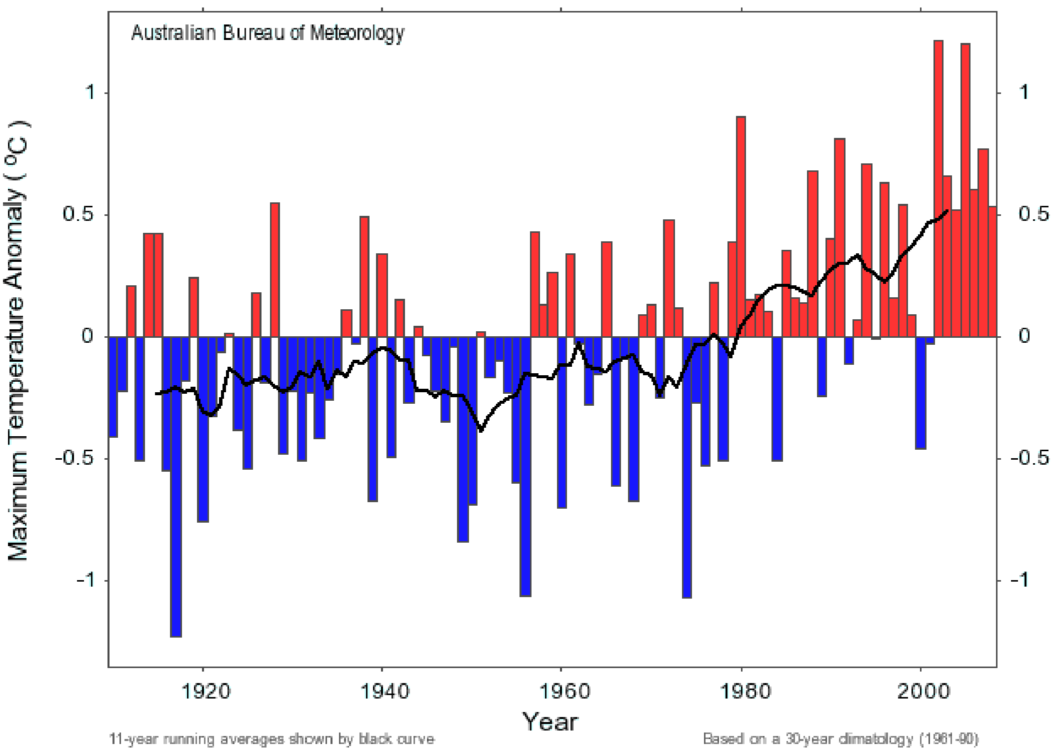


Figure 3-8 Annual Maxima Temperature Anomaly – Australian Mean (BoM, 2009)

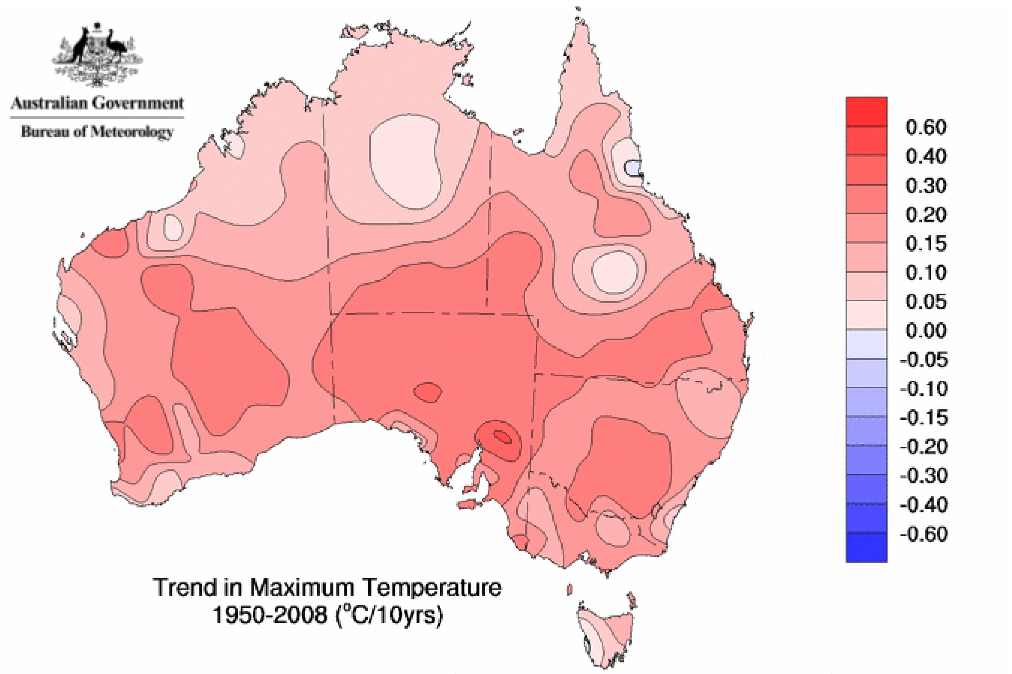


Figure 3-9 Annual Maxima Temperature Trends - Australia (1950-2008) (BoM, 2009)

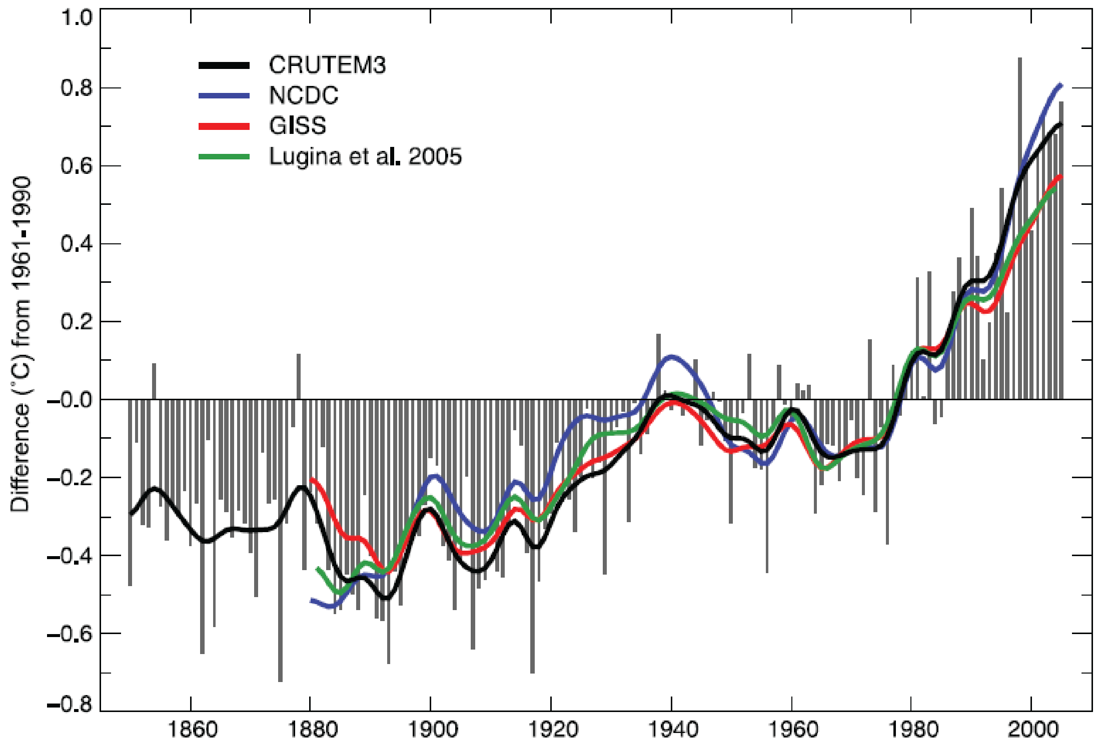


Figure 3-10 Annual Mean Temperature Anomaly – Global (Trenberth *et al.*, 2007)

3.3 Sediment Transport

Historically, developments in coastal engineering have largely been driven by factors such as national defense, agriculture, navigation, economic development, recreation, and the environment (Lochart and Morang, 2002). Today, more so than ever, continuing development pressure within the coastal zone is forcing local, state and national governments to consider the risks relating to issues within this geographical area. It is perceived that future climate change may cause significant stress to coastal systems. Shoreline sediment processes, ideally erosion, resulting from sea level rise and variation in wave climate, represent one of these at risk areas.

The management of erosion is not a new issue. Shoreline change represents one of the most common coastal engineering issues managed in the past, sometimes successfully and other times not. Managing erosion can be difficult due to the complex and varied physical forces driving sediment transport and erosive processes.

Historically, erosion has sometimes resulted from human activities, such as port developments. Alternatively, natural processes, such as storm events or natural sediment deficiencies, have also resulted in shoreline erosion.

In the context of this project, methodologies have been developed capable of assessing the possible impact of climate change driven changes in sea level and wave climate on shoreline erosion. These methodologies have been used to assess the possible impacts of climate change on the beach system at Wooli Wooli in northern NSW.

Apart from shoreline change resulting from human activities, changes to current statistical distributions representing the following forcing parameters may result in shoreline change:

- Increases in sea level;
- Changes in wave height, direction and period; and
- Changes in storm intensity and frequency.

Future climate change has the potential to result in changes to all of these forcing parameters.

Predicting shoreline response to shifts in these parameters cannot be done using a single approach or model. Depending on the forcing function, the timescale of shoreline response may be short-term, in the order of hours (eg. storm erosion); medium-term, taking weeks to months (eg. longshore sediment gradient response) or long-term, possibly taking years (eg. sea-level rise).

To account for the variability in forcing functions and large ranges in response timescales, a combination of dynamic modelling approaches has been developed and applied to predict possible shoreline response resulting from climate change.

3.3.1 Assessment Approach

As stated previously, managing erosion can be difficult due to the complex and varied physical forces driving sediment transport and erosive processes. In reality, broken and non-broken waves in the nearshore combine with various horizontal and vertical patterns of nearshore currents to transport beach sediments. Sometimes this transport results in a local rearrangement of sand into bars and troughs (**cross-shore transport**), or into a series of rhythmic embayment cut into the beach. At other times there are extensive longshore displacements of sediments (**longshore transport**), possibly moving hundreds of thousands of cubic metres of sand along the coast each year (Dean *et al.*, 2002). Modelling of sediment transport processes have historically been separated into each of these two components, longshore and cross-shore sediment transport (Rosati *et al.*, 2006).

This split approach outlined by Rosati *et al.* (2006) significantly simplifies many difficulties which may be encountered during long-term sediment transport analysis. During the split approach, longshore and cross-shore sediment transport is calculated independently of one another. At the completion of the model simulation the results from the cross-shore/longshore modelling are summed to calculate the final shoreline position.

Testing undertaken during this study indicates that using the split model approach may be suitable for study areas on open beaches, away from control features such as headlands. In the vicinity of control features, wave refraction and/or diffraction may result in significant variations in wave height for different alongshore locations. The variations in wave height may results in alongshore variations in cross-shore sediment transport and non-linear alongshore erosion volumes. For example, for some pocket beaches, storm events producing oblique incident waves may result in one end of the beach experiencing large waves whilst the opposite end of the beach may be sheltered producing substantially milder waves. The end of the beach experiencing the larger waves is expected to experience greater erosion due to cross-shore process compared to the sheltered end of the beach. During storm recovery periods these variations in cross-shore response, resulting in slight reorientation of a beach, may influence longshore transport gradients. Using the split model approach for long timescales (eg. decadal) will not account for the possible interaction between the longshore and cross-shore sediment transport processes. To manage this issue a methodology linking a dynamic longshore and cross-shore model has been proposed.

Although the modelling approach used to assess the physical impacts of climate change uses a dynamically linked cross shore/longshore model, for readability purposes, the sediment transport processes are being reviewed separately within this literature review.

3.3.2 Longshore Transport

The author is currently not aware of any studies assessing the impacts of climate change on longshore transport. Most existing studies focus uniquely on cross-shore processes. These studies have historically calculated shoreline recession predictions based on predicted values of sea level rise derived by authorities such as the IPCC (Intergovernmental Panel on Climate Change, 2007) or the United States Environmental Protection Agency (Hoffman, Keyes and Titus, 1983). As such it is difficult to make any comments comparing the current research study to previous work in the context of climate change regarding longshore sediment transport.

The lack of climate change studies assessing the likely changes in longshore sediment transport is most probably due to the limited availability of information documenting the predicted changes to offshore wave climates resulting from climate change. Fortunately, this information is now available for the NSW coastline. The results specific to Wooli Wooli Beach and Batemans Bay have been documented in the CSIRO study completed in 2007 (McInnes *et al.*, 2007). Based on the results of the CSIRO study, shifts in wave climate will be used to predict possible longshore transport gradients and associated erosion and/or accretion.

A recent study completed by Ranasinghe *et al.* (2004a) identified possible links between the southern oscillation index (SOI), changes in wave climate and beach rotation/realignment. This study identified the potential impact changes in wave climate resulting from shifts in medium term weather cycles may have on short pocket beaches. The study found that for the two selected locations, Palm Beach and Narabeen/Colloroy Beach, during the El Nino climate phase, the northern end of both beaches accreted whilst the south end of the beaches eroded. This resulted in a net clockwise rotation of the beach. During the La Nina phase however, the opposite occurred, resulting in a net anticlockwise rotation of the beach. Assessment of the wave climate produced by both weather patterns found the El Nino periods were typically milder with more southeast/south incident. In comparison the waves during La Nina periods produced more energetic and more east/northeast incident waves. These trends produce different wave climates for both SOI phases, which in turn drove the rotation of beaches.

In the climate change perspective, the IPCC predict that climate change is likely to result in future tropical cyclones becoming more intense and extratropical storm tracks moving poleward (Meehl *et al.*, 2007). These climatic shifts will result in modified wave climates for coastlines worldwide compared to the present. This assumption is supported by the results of the CSIRO study (McInnes *et al*, 2007) focusing directly on the changes in wave climate for the NSW coastline

The results of the Ranasinghe *et al.* (2004a) study emphasis the need to include longshore sediment transport calculations in beach erosion studies assessing the impacts of climate change. This accounts for the possibility of long-term shifts in wave climate resulting in shoreline rotation/realignment.

3.3.2.1 Longshore Sediment Transport Theory

Waves breaking obliquely to a shoreline produce a longshore current up or down coast depending on the direction of the incoming waves. The movement of sediment resulting from this longshore current is termed longshore sediment transport or littoral transport. The term littoral drift refers to total volume of sediment being transported for a defined period.

The subject of total longshore transport has been studied for approximately 5 decades. Even so, there is significant uncertainty regarding certain aspects of this transport component, including:

- The effect of grain size;
- Barred topography and
- Cross-shore distribution of longshore transport (Dean *et al.*, 2002).

Furthermore, sediment moved along a coastline under the action of waves and longshore current is transported via several modes: bed, suspended and swash load transport. It is not entirely clear which of these motions predominates for various wave conditions, sediment types and locations on the profile or even whether it is important to distinguish between the different mechanisms (Dean and Dalrymple, 2002). To further complicate things, unfortunately, there is no accurate meter or gauge that can measure long-term littoral drift. The total transport is often estimated by such measures as the impoundment of sand at a jetty or breakwater or the deposition of sediment in an inlet or harbour. (Dean and Dalrymple, 2002). However, it should be noted that the accuracy of these indirect measurements depends directly on the efficiency of the coastal structure at trapping sediment.

Impoundment options are not available at all locations, alternatively the trapping efficiency of a given structure may not be sufficient for research purposes. Due to these issues some studies have employed the use of sediment tracer experiments to measure short-term longshore transport rates and beach change. Many engineers consider sediment tracer experiments to be the only correct method the measure longshore transport (Huchzermeyer, 2005).

Both of the above-mentioned methods have been used to develop the longshore transport theories and empirical formula commonly used to calculate littoral drift. To obtain realistic results when using these empirical formula, where possible, it is encouraged to use any available site specific calibration data to optimise the input parameters. The writer believes, if available, impoundment loadings should be used to define calibration parameters as they account for total transport quantities moving alongshore (The sum of bedload, suspended load and swash load transport) for longer periods than are generally available using tracer techniques.

Computationally there are various engineering techniques available to estimate littoral drift. Two of the most commonly used longshore sediment transport equations are the CERC and Queen’s formula.

3.3.2.2 CERC

Arguably the most widely used model for estimating longshore sediment transport rates is the CERC formula. The model was derived based on the assumption that the total longshore sediment transport rate is proportional to longshore energy flux. (Smith *et al.*, 2003). The CERC formula is given by:

$$Q_l = K\left[\frac{\rho\sqrt{g}}{16\gamma^{\frac{1}{2}}(\rho_s-\rho)(1-n)}\right]H_b^{\frac{5}{2}}\sin(2\alpha_b) \qquad \textbf{Equation 1}$$

Silvester and Hsu (1997) state that the accuracy of CERC formula should be regarded as no better than an order of magnitude estimate. During Silvester and Hsu's assessment, swell parameters were averaged over time to create one representative wave case, to which the CERC formula was applied. It is believed these averaging techniques may have influenced their erroneous results. Simple averaging of long-term wave conditions into a single wave case will not provide an accurate representation of the energy weighting associated with the different directional components of a swell over a long sample period. As a minimum, if this "averaged approach is to be used, since sediment transport is proportional to $H_b^{5/2}$, the mean of $H_b^{5/2}$ should be applied rather than H_b.

To obtain more accurate results, the timescale of the CERC formula calculations should be equal to the timescale of the input wave data. The net sum of the calculated longshore transport volumes is then used to calculate the littoral drift for longer time periods.

Even when using this method to implement the CERC formula, an uncalibrated model using an inappropriate *K* value may produce very erroneous results. One of the main criticisms of the CERC formula is the reliance of the methodology on the dimensionless coefficient *K*. With the correct application of the *K* coefficient as a calibration parameter, greater accuracy (better than ±50%) in longshore sediment transport predictions can be obtained. This has been documented in the various studies testing the accuracy of the CERC formula (Huchzermeyer, 2005; King, 2006; Smith, 2006).

King (2006) comments on the accuracy of the CERC formula using a calibrated and uncalibrated value for the dimensionless coefficient *K*:

> "The limitations of the CERC formula are well known. With adequate calibration, the CERC formula can estimate longshore transport within ±50%. However, without calibration, the CERC formula only provides an accuracy of one to two orders of magnitude (Greer and Madsen, 1978; Fowler *et al.* 1995; Wang *et al.*, 1998). Even so, the CERC equation continues to be useful primarily because of it's simplicity and because of the failure of more sophisticated models to clearly demonstrate substantially superior accuracy relative to the effort required to employ them."

King's comments highlight the need to use the correct *K* value when using the CERC formula. As such, much research has been conducted trying to provide estimates for appropriate *K* values relating to grain size. For grain sizes less than the 1.0mm the Coastal Engineering Manual (Rosati *et al.*, 2006) provides an approximate guide for appropriate *K* values. Figure 3-11 shows the Coastal Engineering Manual data (note the data uses H_{rms} wave heights) (Rosati *et al.*, 2006). For sediment sizes greater than 1.0mm King (2006) completed a thorough literature review and collated the data provided in Figure 3-12. King recommends the use of the curve identified as 'Equation 8' in Figure 3-12.

Review of the raw data used to derive the *K* value relationships with grain size show significant scatter. This further reinforces Rosati *et al.'s* (2006) comment:

> "*While it is generally thought that the K coefficient should decrease with increased grain size, the nature of this relationship is not well understood at present. Again, because of the limited data set and inherent variability in measuring longshore sediment transport rates, predicted K coefficients may vary considerably from appropriate values for any particular site".*

As such the *K* values recommended in Figure 3-11 and Figure 3-12 should only be used as a guide.

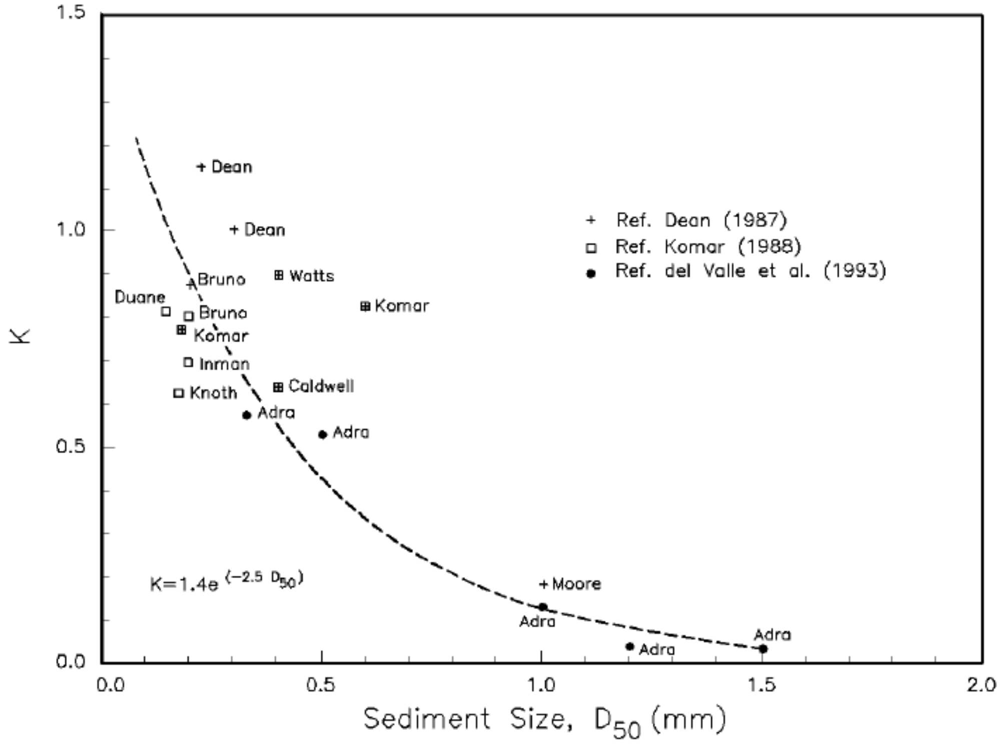


Figure 3-11 Coefficient *K* - Grain Size D_{50} < 1.0mm (Rosati *et al.*, 2006)

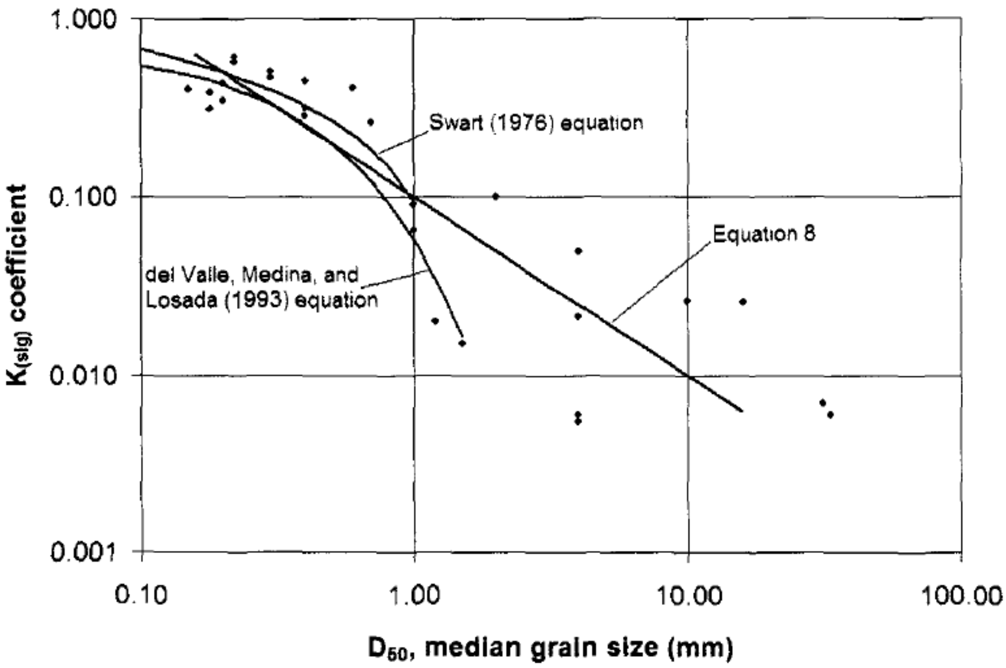


Figure 3-12 Coefficient *K* - Grain Size D_{50} > 1.0mm (King, 2006)

3.3.2.3 Queens Formula

Due to the CERC formula's inadequacies, relating to the formulas dependence on the dimensionless *K* value, various other studies have been undertaken aiming to develop new methods for predicting longshore transport. The most commonly used alternative to the CERC formula is known as the Queens formula, developed by Kamphius (1991). The Queens formula was derived using large wave tank simulations and does not rely on a dimensionless calibration constant. Assuming a sediment porosity of approximately 32% the Queens formula is given by:

$$Q = 6.4 \times 10^4 H_{sb}^2 T_p^{1.5} m_b^{0.75} D_{50}^{-0.25} \sin^{0.6}(2\alpha_b)$$ **Equation 2**

Comparisons between the Queens formula and the CERC equation from various sources (Smith, 2006; Huchzermeyer, 2005) indicate that the Queens formula is favoured verses the uncalibrated CERC equation. However, if the CERC equation is calibrated using recorded/estimated littoral drift volumes, both methods provide comparable results.

3.3.2.4 Bayram *et al.*

Bayram *et al* (2007) derived an equation capable of calculating longshore transport volumes resulting from a combination of wave, wind and tide driven currents. Hindcast validation testing has shown that the inclusion of wind and tide driven current inputs result in increased longshore transport estimate accuracy (Bayram *et al*, 2007). For the purpose of this study however, focused on future climate change, the inclusion of these additional parameters significantly increases the level of assessment complexity and also level of uncertainty associated with the additional boundary condition inputs. Bayram *et al* comment that excluding the wind and tide driven forcings reduces the the Bayram *et al* (2007) approach to be exactly equivalent to the CERC formula.

It is recognised that both the CERC, Queens and Bayram *et al*. formula are simplified representations of extremely complex environmental systems. In the absence of other available calculation methods, and acknowledging the preference not to include wind input within the future climate change assessments due to the author's familiarity with the CERC formula, it will be applied during the research project.

3.3.2.5 Shoreline Evolution Modelling

Numerical shoreline evolution modelling, which uses a fixed height of the active profile, also known as one-line modelling is a well known tool in Coastal Engineering. One-line modelling can be used to predict changes in coastline position due to gradients in the longshore sediment transport rate. Hanson and Kraus (1989) describe one-line modelling as an automated means to perform a time-dependent sediment budget analysis. GENESIS, standing for the GENEralised model for the SImulation of Shoreline change represents one of the original one-line models.

One-line models are typically developed representing the shoreline as a single line. Offshore, parallel contour assumptions are used to represent the bathymetry required to transform deepwater inputs into the nearshore zone. Shoreline change is calculated due to spatial and temporal differences in longshore transport as produced by breaking waves (Rosati *et al.*, 2006). Figure 3-13 provides a simplified diagram representing a one-line model.

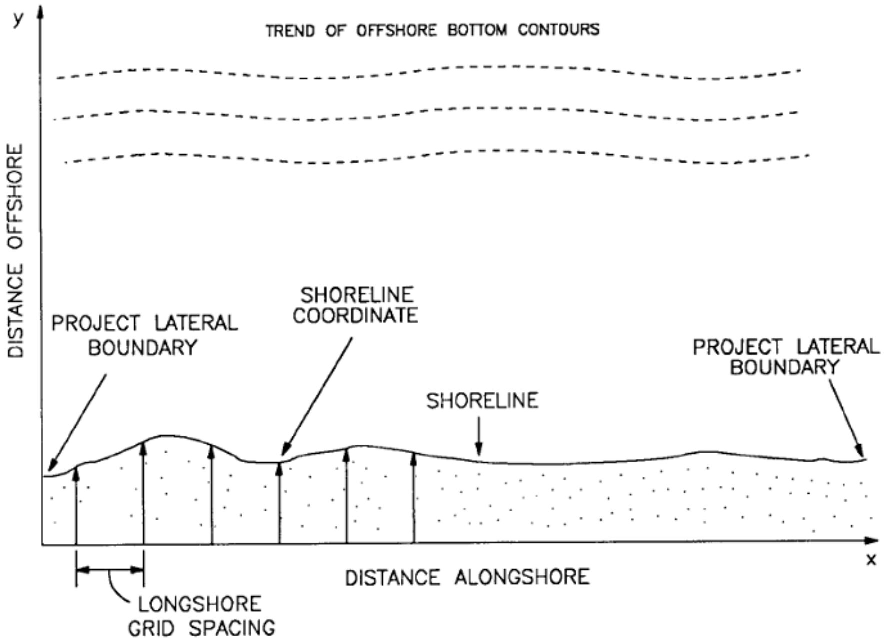


Figure 3-13 One-Line Model Representation (Rosati *et al.*, 2006)

Numerically, GENESIS is based on a finite difference scheme; because of its one-dimensional nature there are a number of simplifying assumptions that have been used to create the shoreline change model (Dyke, 2007). Hanson and Kraus (1989), one of the developers of GENESIS, lists the assumptions made by GENESIS. These include:

- The beach profile shape is set to be constant, based on the $Ax^{2/3}$ profile proposed by Bruun (1954) and Dean (1977). This assumption is based on the theory that beach profiles maintain an average shape that is characteristic of the particular coast. Although seasonal changes in wave climate cause the position of the shoreline to move shoreward and seaward in a cyclical manner, with corresponding change in shape and average slope of the profile, the deviation from the average beach slope over the total active profile is relatively small. If it is assumed that profile shape does not change, any point on the profile is sufficient to specify the location of the entire profile with respect to a baseline, which in this case is the shoreline. Thus, one contour line can be used to describe changes in the beach plan shape and volume as the beach erodes and accretes. Figure 3-14 shows an example of shoreline erosion using a fixed beach profile.

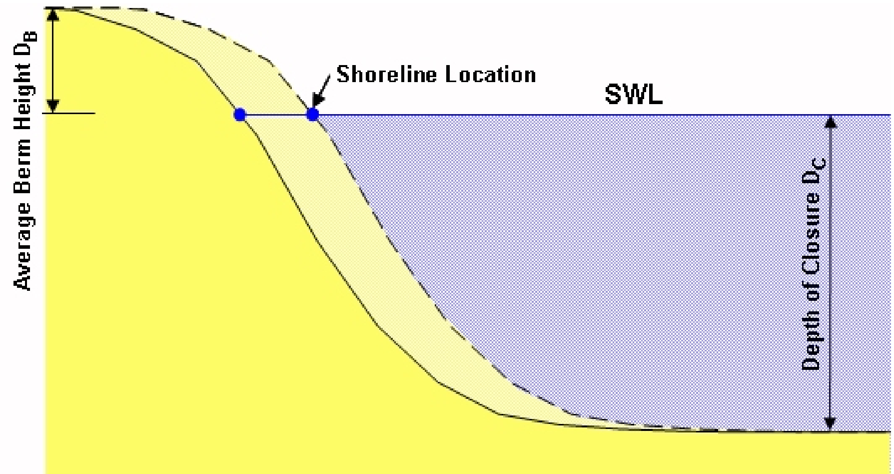


Figure 3-14 Shoreline Change Using a Fixed Profile (Jackson 2007)

- The shoreward limit (berm) and seaward limits (depth of closure) of the profile are constant. Restriction of profile movement between these two limits provides the simplest way to specify the perimeter of a beach cross-sectional area by which changes in volume, leading to shoreline change, can be computed (Jackson, 2007). Figure 3-15 represents volume calculation methods employed by GENESIS limited by the defined berm height and depth of closure.

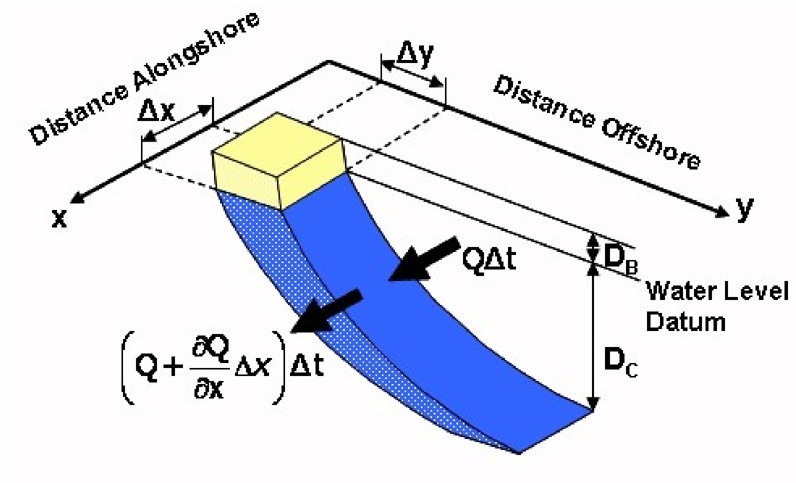


Figure 3-15 GENESIS Volume Calculations (Jackson 2007)

- Model boundaries for open coast beach models assume transport into the open boundary of the model is a function of the breaking waves height and direction alongshore. Nearshore current behaviour is not entered into the model.

- It is assumed that there is a clear long-term trend in shoreline behaviour. This assumption ensures there is a steady signal of shoreline change averse to the "noise" in the beach system produced by storms, seasonal change in waves, tidal fluctuations and other cyclical and random events.

Hanson and Kraus (1989) comment that the assumptions listed above define a flexible and economical shoreline change simulation model that has been found applicable to a wide range of coastal engineering problems.

Hanson and Kraus' comment is valid as long as the modeller understands the implications of the above assumptions and uses GENESIS within the limits it was designed for. In short, the author believes the follow recommendations should be made:

- It should be recognized that the parallel contour assumption extends beyond the depth of closure. The offshore contour orientation, upon which the incoming waves are refracted, is calculated as a smooth rendering of the shoreline orientation (Jackson, 2007). If bathymetric features beyond the depth of closure result in non-uniform wave conditions (due to refraction or diffraction) in different sections of the model a nested wave transformation model should be used to input wave conditions into the model. GENESIS supports this capability

- The depth of closure and berm height may vary alongshore. The model domain should be carefully placed to avoid large discrepancies between real and modelled values.

- GENESIS should only be used if breaking waves are the dominant mechanism for transport alongshore.

Regarding climate change, one-line models are unable to directly assess the impact which sea level rise may have on shoreline evolution. Shoreline recession associated with sea level rise is complex, primarily being driven by cross shore sediment transport processes, though also being influenced by longshore processes where headland controls are present. The current model structure of GENESIS is unable to accommodate for these combined processes.

Due to this, as part of this project, a one-line model based on the CERC formula (Equation 1), titled LSMOD, has been developed. The details of this model are outlined in Section 4.2. The developed one-line model has been dynamically linked to a cross-shore model, XSMOD, to represent the cross/ longshore sediment transport processes mentioned above.

3.3.3 Cross-Shore Transport

Cross-shore sediment transport encompasses both offshore transport, such as occurs during storms, and onshore transport, which dominates during milder wave activity. As shown in Figure 3-16, there are numerous techniques available for modelling cross-shore sediment transport. Each of the listed methods is suited towards specific types of cross-shore sediment transport problem. This is often defined by the desired accuracy of the model, the forcing parameter, and the shoreline response duration being modelled.

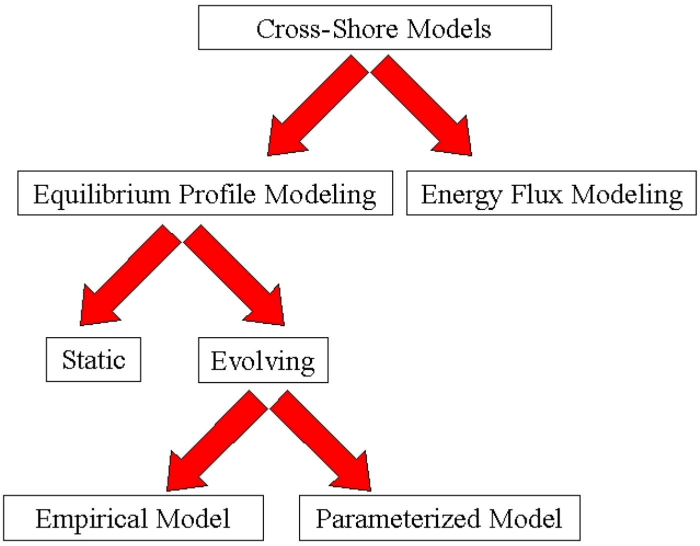


Figure 3-16 Cross Shore Modelling

It is recognised that cross-shore sediment transport occurs for the full range of timescales. On the short-term, storm events may cause significant erosion. For any particular location, shoreline erosion typically experienced during storm events is dependent on the following three forcing parameters:

- Magnitude of wave energy increase relative to average swell conditions;
- Magnitude of water level increase (Storm Surge); and
- Storm Duration

During storm events, wave induced erosion, driven by increased water levels and wave energy are typically time limited. Although the shoreline responds quickly during the storm events, often the final erosion volume only represents a portion of the total possible erosion. Figure 3-17 illustrates how the duration of a storm event may not result in the maximum potential erosion for a given wave height and elevated water level. In the illustrated example, the shoreline receded approximately 60% of the possible maximum recession had the storm duration been sustained for an indefinite period of time.

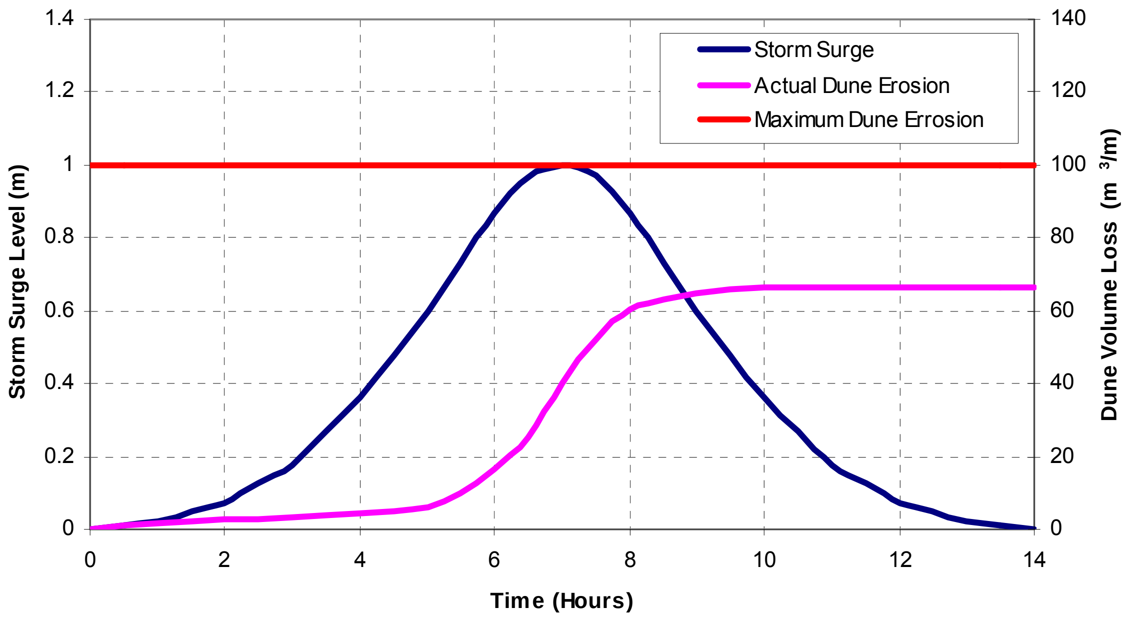


Figure 3-17 Cross-Shore Profile Response

In comparison, climate change driven sea level rise induced erosion is not limited by time. Sea level change occurs gradually over long timescales (decades to centuries). Equally, the timescale of profile change resulting from climate change driven sea level rise occurs over long time scales. In this case the erosion volume is limited by the increase in water level, instead of by time. This is due to the fact that the new water level will remain at the new level for time periods longer than the erosion response timescale. In this case the erosion is likely to match the maximum possible erosion volume resulting from the increase in water level.

For this climate change study it is important that the selected modelling approach is able to represent the cross-shore evolution for both the short and long-term processes. Based on the defined model types presented in Figure 3-16 a variety of the available cross-shore models will now be discussed.

3.3.3.1 Energy Flux Models

Detailed process based models represent theory predicting profile formation based on the motion of individual sand particles (suspended load and bedload) driven by applied forces (wave or current). Although this is an attractive approach from the standpoint of completely comprehending sediment transport processes, as various authors note, currently it appears to be beyond our present state of knowledge (Dean and Dalrymple, 2002; Larson, 1989, Miller and Dean, 2004). Maddux *et al.* (2006) writes about the need for improved accuracy in sediment transport models.

"Models of cross-shore sediment transport have in recent years moved away from empirical models of profile evolution and towards process-based models. Most of these models incorporate recent advances in nearshore hydrodynamic models, including those based on time-averaged equations (Van Dongeren and Svendsen, 2000; Shi et al. 2003), Boussinesq approximations (Rakha et al., 1997; Kennedy, et al., 2000; Lynett et al. 2002; Herbers et al., 2003); the nonlinear shallow water wave equations (Kobayashi, 1999); Reynolds-averaged Navier-Stokes solvers (Lin and Liu 1998); and direct numerical simulations (Slinn and Riley, 1998). These improvements have led to our ability to estimate the free water surface with accuracies of about ±20% and near-bottom velocities

with about ±50% accuracy. Meanwhile, predictions of sediment transport processes in the nearshore have not been met with the same successes. For example, estimations of instantaneous sediment suspension rates are still off by at least a factor of 2-3, even with sophisticated LES models (Zedler and Street, 2002)."

Far from being a criticism of the existing models, this is a testament to the extreme difficulty in representing all of the complex physical interactions within the surf zone, which result in highly three-dimensional nearshore morphology. Due to the inherent inaccuracies using energy flux models and the large computational demand required to run the models, current energy flux models are not suitable for long-term (>50 years) cross-shore sediment transport assessments.

3.3.3.2 The Cross-Shore Equilibrium Profile

Based on studies undertaken in Denmark and Monterey Bay, California, Bruun (1954) was the first to propose the use of a generic equation as a method to represent the cross-shore profile. Many dynamic cross-shore models have since used this method as a simplified method to represent the cross-shore profile. Before discussing the available cross-shore models it is worthwhile first discussing the derivation and accuracy of the various available profile equations.

The profile equation proposed by Bruun (1954) is represented by:

$$y = Ax^{2/3}$$ **Equation 3**

In addition to offshore distance, represented by *x*, the sediment scale parameter, *A*, is required to calculate the bed elevation using the $Ax^{2/3}$ equation. Dean *et al.* (2002) summarises the various approaches, some of which are based on empirical estimates relating to sediment fall velocities others based on dimensional analysis. Based on the Dean *et al.* assessment, the recommended approach for sediment scale parameter selection is based on research completed by Dean (1987) and Moore (1982). Dean and Moore developed an equation to calculate the sediment scale parameter using a least squared fit of the $Ax^{2/3}$ equation compared with measured real profiles. Independent testing by the writer using the various sediment scale parameter approaches found the method recommended by Dean *et al.* to be the most accurate. Additional work done by Pruzak (1993) also supports the theory based on the least squared fit of real profiles. Figure 3-18 graphically compares the sediment scale parameter estimation approaches, as shown by Dean *et al.* (2002). In Figure 3-18 the line identified as the "suggested empirical relationship" represents the approach recommended by Dean *et al.*

Using the correct sediment scale parameters, Dean verified the $Ax^{2/3}$ profile proposed by Bruun for over 500 beaches across the United States (Dean and Dalrymple, 2002). The writer has also tested the equation using numerous recorded profiles measured from the Gold Coast, Australia, with satisfactory results.

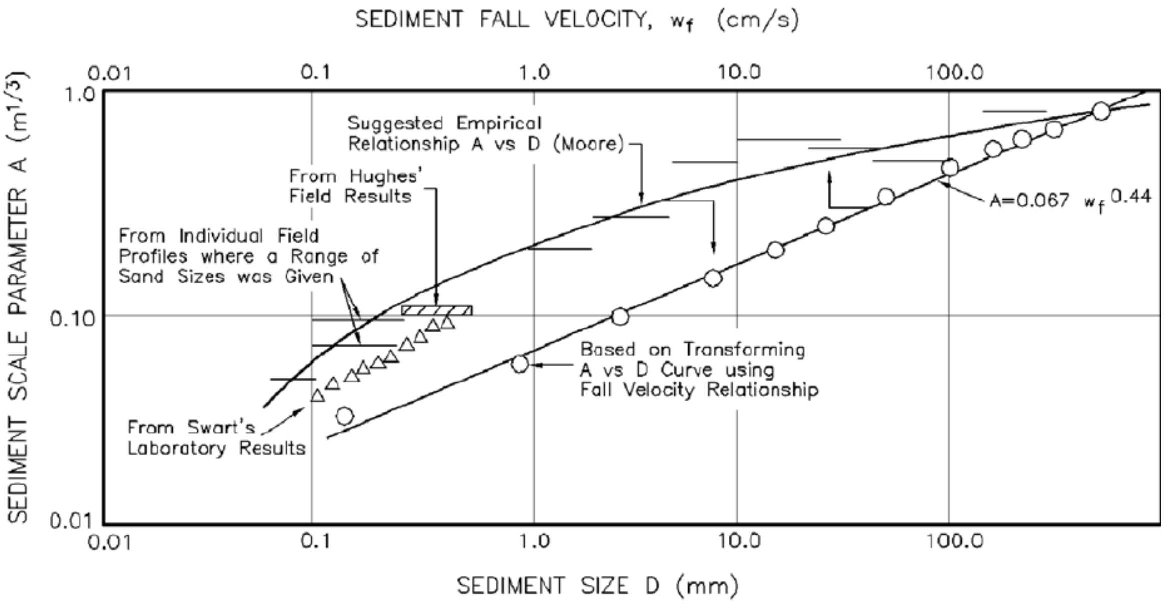


Figure 3-18 $Ax^{2/3}$ Sediment Scale Parameter (Dean et al, 2002)

Since the initial derivation of the $Ax^{2/3}$ theory, other researches have developed comparative profile equations.

Dean and Dalrymple

Dean and Dalrymple (2002) extended the profile equation derived by Bruun (1954) to include the effect of wave setup. The equation proposed by Dean and Dalrymple is given by:

$$y = \frac{Ax^{2/3} - \overline{\eta}_b - \mathrm{K}h_b}{(1-\mathrm{K})} \qquad \textbf{Equation 4}$$

where:

$$\mathrm{K} = \frac{3\gamma^2 / 8}{1 + 3\gamma^2 / 8} \qquad \textbf{Equation 5}$$

Vellinga (1982; 1983) derived a profile equation using large scale wave flume tests whilst developing theory to predict shoreline erosion resulting from storm conditions. Vellinga's profile equation is represented by:

$$(7.6/H_0)y = 0.47\left[(7.6/H_0)^{1.28}(w/0.0268)^{0.56}x + 18\right]^{0.5} - 2.00 \qquad \textbf{Equation 6}$$

Comparisons between Vellinga's and Dean and Dalrymple's profile equations using surveyed post storm profiles from the Gold Coast have shown both plots are very similar in nature, shown in Figure 3-19. The assessment did not identify a favoured approach (both methods produced near identical profile plots). As neither method is computationally more accurate the author has chosen to use the Dean and Dalrymple's profile equation instead of Vellinga's due to the simplistic arithmetic nature of the equation. For the remainder of this document the Dean and Dalrymple profile equation will simply be referred to as the $Ax^{2/3}$ profile.

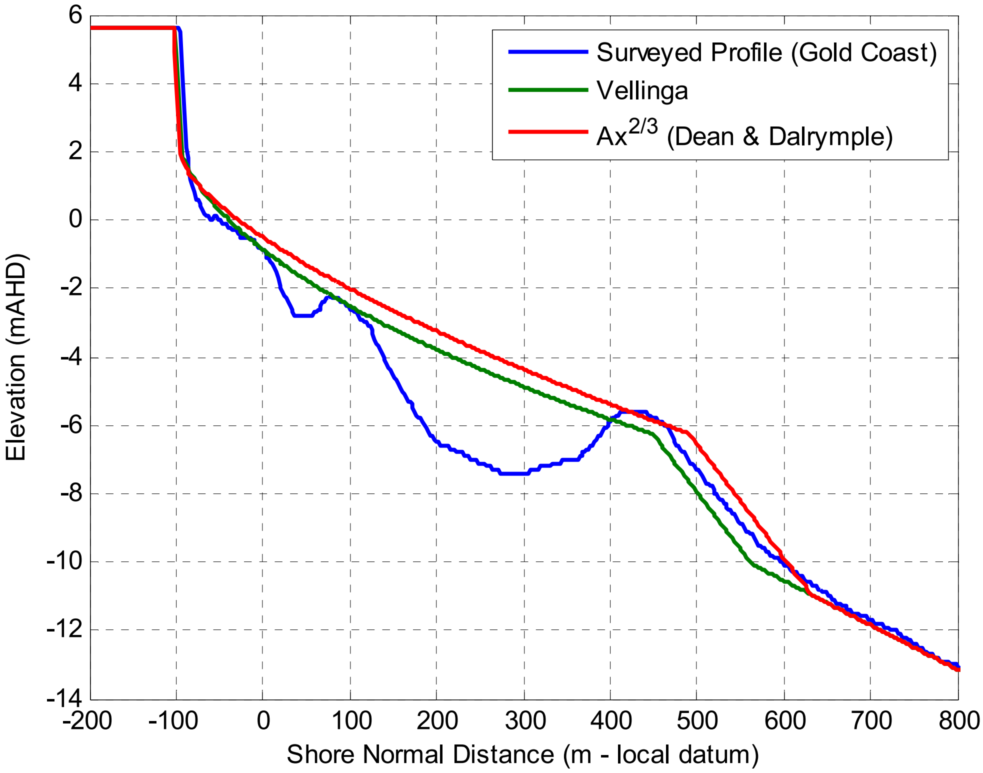


Figure 3-19 Profile Equation Comparison

One of the shortcomings of these profile equations is their inability to represent barred profiles, which is a common feature of NSW beaches. Assessment of the available predictive cross-shore change models (eg. Kriebel and Dean (1985); Miller and Dean (2004)), which utilise the $Ax^{2/3}$ profile, identified the inherent assumption within all approaches, stating that conservation of mass within the profile is required. These theories will be discussed in detail in following sections. However, this assumption highlights the need, when representing real recorded profiles, that a mass balance approach be applied when calculating a representative initial $Ax^{2/3}$ profile. This is especially important for barred profiles where the nearshore gutter will often lie below the plotted bed level for the representative $Ax^{2/3}$ profile. Figure 3-20 shows an example of a real survey profile from the Gold Coast plotted against a representative $Ax^{2/3}$ profile, after mass balance calculations have been completed.

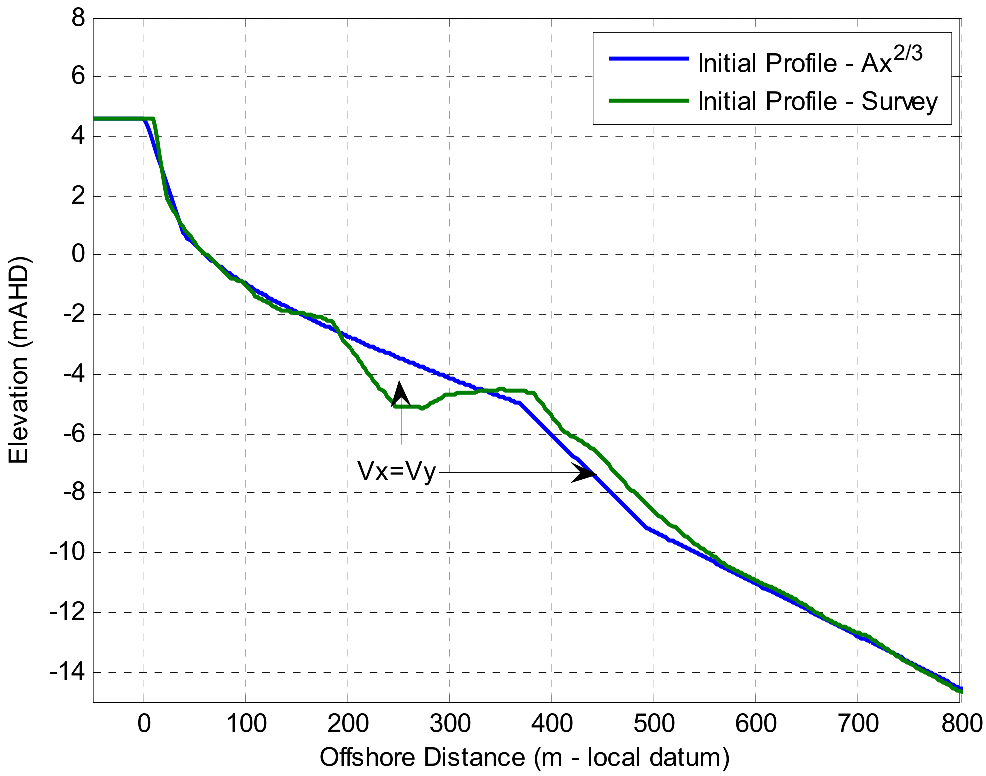


Figure 3-20 $Ax^{2/3}$ Mass Balance Profile

As Figure 3-20 shows, the offshore location of the outer bar is brought shoreward slightly in the $Ax^{2/3}$ profile to account for the displaced sediment from the nearshore gutter. The shoreward translation of the breakpoint is calculated to ensure equal profile sediment volumes for the $Ax^{2/3}$ profile and the survey data. To achieve this, the value for Vx is calculated to equal Vy in Figure 3-20.

Many of the methods which use the $Ax^{2/3}$ profile do not use "actual" profile surveys to represent the cross shore profile. Instead the models use the equilibrium profile represented by a derived $Ax^{2/3}$ profile. The equilibrium beach profile represents a statistical average profile which maintains its form apart from small fluctuations including seasonal fluctuations.

3.3.3.3 Static Response Equilibrium Profile Models

Static models are represented by formulas which are used to define potential shoreline recession based on fixed input parameters. Static models are not able to provide shoreline information for intermediate timesteps between the initial and final predicted shoreline positions.

The Bruun Rule

Following the development of the $Ax^{2/3}$ profile, Bruun (1962) developed one of the first theories capable of predicting shoreline recession resulting from sea level rise. Known as the Bruun rule, this method has historically been used to predict potential shoreline recession resulting from increases in water level. The Bruun rule is represented by the following equation:

$$s = l\frac{a}{h} \quad \textbf{Equation 7}$$

Bruun (1986) describes the application of the Bruun rule. Given an equilibrium beach profile, "a statistical average profile which maintains its form apart from small fluctuations including seasonal fluctuations", a rise in sea level would be followed by:

1. A shoreward displacement of the beach profile as the upper beach is eroded;
2. Movement of the material eroded from the upper beach would be equal in volume to the material deposited on the near offshore bottom;
3. A rise of the near offshore bottom as a result of this deposition, equal to the sea level rise

Figure 3-21 graphically shows the application of the Bruun Rule.

Since its development the Bruun rule has been tested by many researchers against both laboratory and field data, Rangasinghe *et al.* (2007) lists the following studies, Schwarz (1965,1967), Dubois (1975), Rosen (1978), Everts (1985) and Pilkey and Davis (1987). SCOR (1991) comment that although these studies confirmed the basic concept of the Bruun rule concerning the landward and upward shifting of the cross-shore profile in response to a rise in sea level, none have convincingly validated the Bruun rule based on predictions of the magnitude of shoreline recession. As such SCOR recommend the use of the Bruun rule only as an order of magnitude estimate of potential recession.

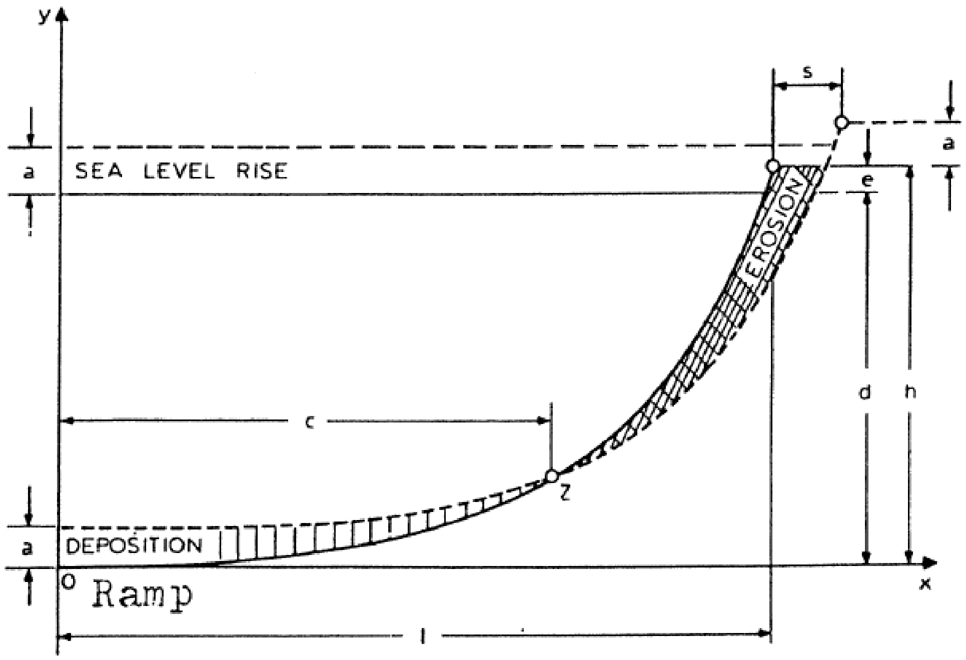


Figure 3-21 The Bruun Rule (Bruun, 1986)

A more recent study by Zang et al (2004) produced more favourable results using the Bruun rule. Zang predicted shoreline recession for a number of locations on the east coast of the USA given data for the last 200 years. Zang et al account the success of his study to correct site selection and more accurate datasets than other previous works. Critics of Zang's work, however, comment that under present sea level rise rates (1-2mm/year) the coastal recession due to other processes such as storms, aeolean transport and gradients in littoral drift may be more dominant than the shoreline recession resulting from sea level rise. If sea level rise rates were to increase in coming years it is, however, likely that it will become the dominant erosive process (Stive, 2004).

Research by Hands (1983) is more promising. Hands tested the Bruun rule using measured data from Lake Michigan, which experienced a 0.2m rise in water level over a 7-year period in the early 1970s. The rate of water level increase, relative to the low energy wave climate experienced in the lake, ensures change in water level is the dominant erosive force. Hands' research predicted shoreline recession using the Bruun rule to 10% accuracy of the measured recession.

Historically, climate change studies, including those completed by Titus *et al.* (1985), Port of Melbourne Authority (1992) and Scharples (2004) have used the Bruun rule, or variations there of, to predict shoreline recession. Within Australia, Rangasinghe *et al.* (2007) notes that most states advocate the use of the Bruun rule for site-specific estimates of recession due to sea level rise in a prescriptive manner. Meanwhile, in contrast, in parts of Europe the Bruun rule is applied with extreme caution. Why is this the case, and is the caution shown by the European countries warranted?

One of the major criticisms of the Bruun rule is the fact that it is too general to be applied to the majority of beach cases. Some of the shortcomings of the Bruun rule include:

- The Bruun rule is only applicable for "equilibrium" beach profiles "a statistical average profile which maintains its form apart from small fluctuations including seasonal fluctuations".
- The Bruun rule does not account for 3D variability such as gradients in littoral drift.
- The Bruun rule assumes no losses of sediment from the active profile (ie. beyond the depth of closure or onshore due to aeolian processes or over wash).
- The Bruun rule does not accommodate variations in sediment characteristics across the beach profile (ie. Sediment within the beach berm is assumed to be of similar character to that in the offshore zone).
- The Bruun rule is also not suitable for beaches exhibiting bedrock outcrops due to possible sediment budget shortages in the eroding berm.

Issues related to defining the upper limit of the active profile and the depth of closure (the offshore limit of sediment transport) can also significantly influence the accuracy of shoreline recession calculations based solely on the Bruun rule.

It is in part due to these limitations that numerous engineers and scientists have attempted to modify and improve upon the Bruun rule. Voice et al (2006) list the following variations to the Bruun rule:

- Cowell and Thom (1994): Produced a model for shoreline response allowing for sea level rise and variation in sediment availability within sandy barrier-dune complexes
- Cowell *et al.* (1995): Used a morphological behaviour model to simulate large scale coastal change.
- Hennecke *et al.* (2004): Used GIS techniques to combine real world examples with models and illustrate the projected costs of sea level rise.

Considering the limitations of the Bruun rule listed above and the varied results from numerous peer reviews, there seems to be a large amount of uncertainty associated with any predictions using the Bruun rule approach in most “real world” cases. As such, in support of the SCOR (1991) recommendations, the Bruun rule should only be used as an initial estimate of shoreline retreat.

As already stated, historically most climate change studies use the Bruun rule to estimate shoreline recession. Within Australia, most states have completed large-scale (often state wide) coastal vulnerability studies assessing the risk which climate change induced sea level rise poses. As a first pass approach, Bruun rule style assessments are suitable as a method to identify vulnerable sections of coastline. If, however, detailed economic assessments predicting the dollar value cost of shoreline retreat are required, a Bruun rule assessment does not provide reliable enough results and, as such, is not suitable.

Edelman Approach

One of the original dynamic cross-shore models was developed by Edelman (1972). Edelman’s model represented a modified version of the Bruun rule accounting for time varying changes in water level, representative of a storm surge to calculate a final shoreline location for storm events. Unfortunately the Edelman theory assumed that for any given time, the profile evolution matches the current storm surge water level. In reality, there is a lag between the maximum shoreline response and the associated elevated water level. Due to this flaw in the Edelman theory, calculations based on the approach typically over predict erosion volumes during storm events. As such, the Edelman approach is not being applied as part of this study.

Vellinga Approach

Vellinga (1982, 1983) also developed a quasi-steady state theory for predicting post storm erosion based on large wave flume tests. Unfortunately Vellinga’s theory is only accurate for constant storm surges with a five hour duration. Vellinga did provide correlation factors for storm surges longer than five hours, though these factors seemed somewhat “ad hoc” (Dean, 1985). The Vellinga approaches inability to accurately predict dynamic variations in water level over varying time scales deem the method not suitable for long-term climate change assessments.

In addition to the limitations already mentioned regarding the application of the Bruun rule, Edelman and Vellinga approaches, none of the static methods discussed can predict long-term shoreline change driven by changes in wave climate accurately. This study is assessing the combined impact of sea level rise and changes in wave climate, requiring a dynamic modelling approach, able to represent both short and long-term cross-shore processes. Evolving cross-shore models attempt to represent both of these scenarios, which make them potentially well suited to this climate change assessment.

3.3.3.4 Evolving Cross-Shore Models

Various evolving cross-shore models have been developed. Initially developed as a tool to predict storm erosion resulting from elevated water levels in combination with increased offshore wave energy the models provide the best available opportunity for climate change studies.

Although most of the models are not appropriate for long-term climate change studies for numerous reasons, a history of the different modelling approaches has been provided. This has been included as it is often the case that theories or alternatively, inadequacies of older models have helped the development of newer models. The evolving cross-shore models discussed below include:

- Kriebel and Dean (1985);
- Kriebel and Dean convolution model (1993); and
- Miller and Dean (2004).

Kriebel and Dean - 1985

Based on the $Ax^{2/3}$ profile theory, Kriebel and Dean (1985) developed a promising predictive storm erosion model. The method utilises an implicit, double sweep procedure to determine the change in position of elevation contours within the profile (Kriebel and Dean, 1985). Similar to the Bruun Rule, the model employs conservation of mass principles. Kriebel and Dean's model differ from the previous models, listed above, in that the method is based on:

1. A numerical solution of simplified equations governing beach profile evolution, including physical mechanisms defining the net cross-shore transport in the surf zone.
2. The complete time history of the storm surge, such that time-dependent profile response is estimated; and
3. The $Ax^{2/3}$ profile.

During the verification of Kriebel and Dean's model it was qualitatively shown that their methodology was adequately able to reproduce various erosion characteristics which had previously not been represented or validated by other cross-shore models.

Firstly, plotting of shoreline recession over time for an instantaneous increase in water level revealed the shoreline response to be exponential in nature, approaching equilibrium asymptotically. This erosion characteristic supports observations from numerous large-scale laboratory tests (Vellinga, 1982, 1983). While this result is straight forward, it is a significant finding, as it indicates the importance of storm duration in the dune erosion process. Numerical results indicate that the time scales for natural beaches may be in the order of 10 to 100 hours for storm conditions and on the order of 1000 to 10,000 hours for sea level induced erosion or beach recovery following storms (Kriebel and Dean, 1985). During hurricane events this often means only 15%-30% of the maximum erosion potential may result. This is due to time-limited forcings in peak water level.

The exponential shoreline response equation is represented by the following equation:

$$R(t) = R_{\infty}(1 - e^{-\frac{t}{T_s}}) \qquad \textbf{Equation 8}$$

This exponential response is shown by Kriebel and Dean (1985) in Figure 3-22. Figure 3-22 shows a plot representing shoreline recession over time resulting from an instantaneous increase in water level.

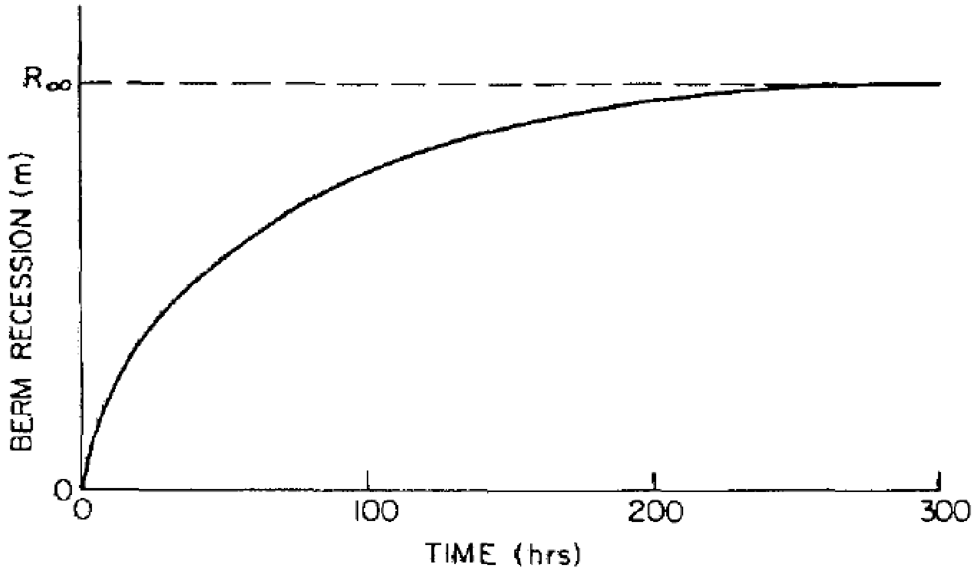


Figure 3-22 Berm Recession Verses Time (Kriebel & Dean, 1985)

Secondly, in accordance with small scale laboratory tests (Hughes and Chiu, 1981) assessing the driving forces of erosion, the model qualitatively predicts increases in water level as the dominant erosion forcing compared with increases in wave energy, resulting from increased wave heights. Figure 3-23 shows the dependency of berm recession resulting from increases in water level In comparison, Figure 3-24 shows the dependency of berm recession resulting from increases in wave height. Although it is difficult to directly compare Figure 3-23 and Figure 3-24 without information documenting the probability or average recurrence interval of the plotted storm surge levels and waves heights, a general comparison does highlight that the berm recession resulting from the plotted rises in water level are an order of magnitude greater than those resulting from the increased wave heights.

In lieu of the positive results indicated by the qualitative validation of the model, Kriebel and Dean used an observed erosion event at Bay County, Florida to test the “real world” applicability of the model. In 1975, Hurricane Eloise, one of the most severe hurricanes experienced in the area (at the time of Kriebel and Dean’s work) crossed the Florida panhandle resulting in significant erosion along Florida’s coastline. For the 20 test cases, the Kriebel and Dean model predicted volumetric erosion from 20.8 to 38.4m^3/m. In comparison, observed recession values of 18.3 to 25.1 m^3/m were recorded. Kriebel and Dean note that while the predicted values are somewhat larger than observed recession values, actual post storm profiles reflect partial recovery of the beach face accounting for approximately 5 m^3/m more erosion. Based on these results, Kriebel and Dean (1985) concluded that their “numerical solution accounted for time-dependent erosion in such a way that the predicted magnitude of the eroded volume is in general agreement with field data of average volumetric analysis”.

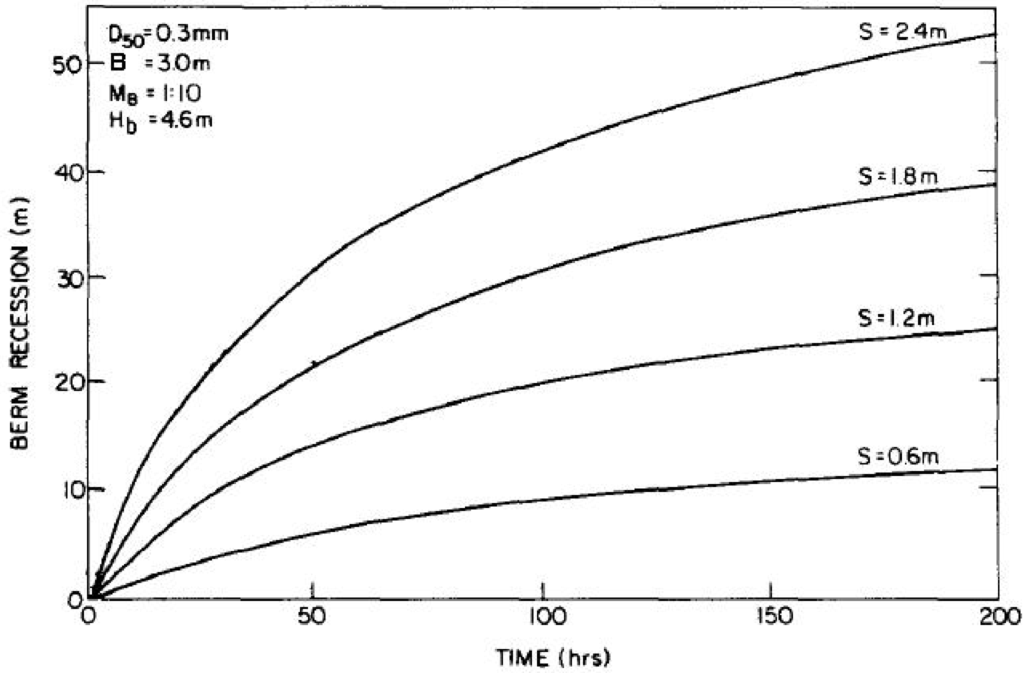


Figure 3-23 Berm Recession vs Water Level (Kriebel & Dean, 1985)

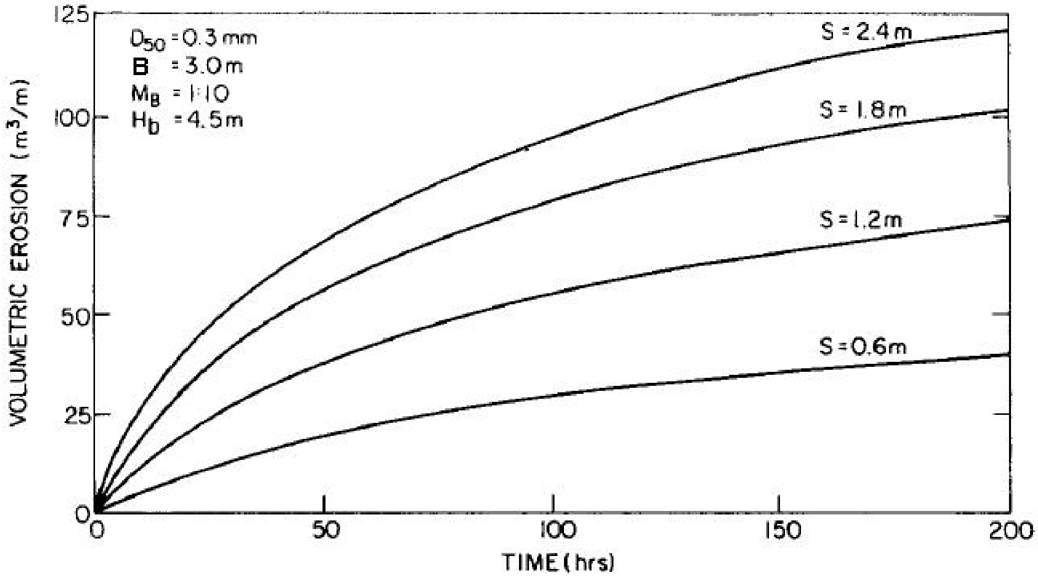


Figure 3-24 Berm Recession vs Wave Height (Kriebel & Dean, 1985)

One of the shortcomings of the Kriebel and Dean model is its inability to represent beach recovery and thus the cumulative effects of successive storms. Unfortunately this makes the Kriebel and Dean unsuitable for long-term assessments such as climate change assessments predicting the impact of potential changes in offshore wave climate.

Kriebel and Dean Convolution Model (1993)

In 1993, Kriebel and Dean developed another evolving cross-shore beach and dune erosion model to supplement their previous model developed in 1985. Drawing on the work undertaken in 1985 the new Kriebel and Dean model assumes the beach as a linear system such that the exponential beach response to erosion forcings is convoluted or folded with a time dependent erosion forcing function (Kriebel and Dean, 1993). This method was an improvement on Kriebel and Dean's previous work (1985), as it provided a simpler method to calculate shoreline recession with comparable accuracies.

To apply the convolution method the following procedures are followed;

1. Based on the initial profile forms, the peak storm surge level, and the estimated breaking depth the analytical solution for the maximum possible erosion potential, R_∞, is obtained. For beach profiles backed by high dunes with no backshore the following equation can be used to calculate R_∞. Figure 3-25 illustrates the input parameters for the below equation.

$$R_\infty = \frac{S(x_b - \frac{h_b}{m})}{B + D + h_b - \frac{S}{2}}$$ **Equation 9**

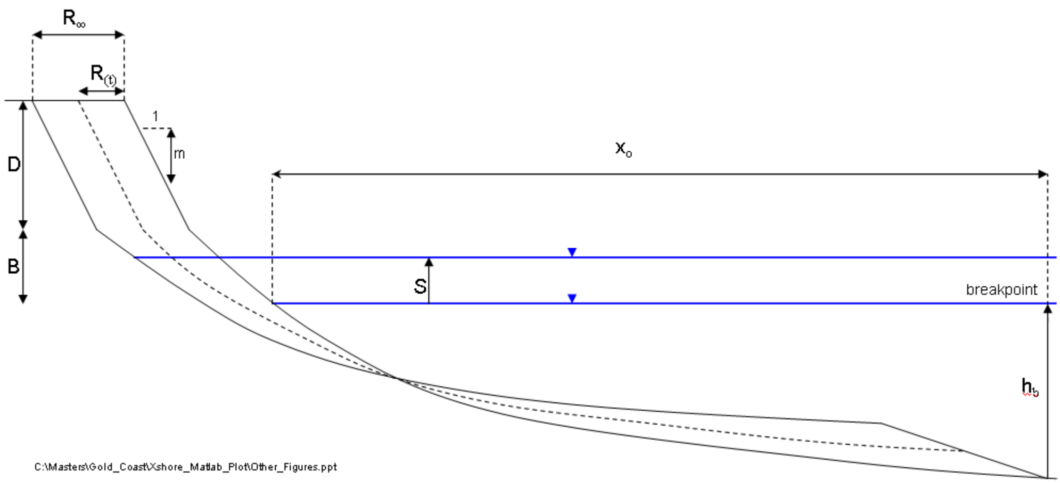


Figure 3-25 Convolution Method Parameter Definition

2. Calculate the erosion time scale for the erosion event using the following equation where H_b represents the maximum breaking significant wave height and A represents the grain size dependent sediment scale parameter.

$$T_s = 320 \frac{H_b^{3/2}}{g^{\frac{1}{2}} A} \left(1 + \frac{h_b}{B} + \frac{m x_b}{h_b} \right)^{-1}$$ **Equation 10**

3. Calculate the erosion time scale to storm duration (T_D) ratio.

$$\beta = 2\pi \frac{T_S}{T_D}$$ **Equation 11**

4. Calculate the shoreline position at time *t* where $\sigma = \pi / T_D$

$$\frac{R(t)}{R_\infty} = \frac{1}{2}\left\{1 - \frac{\beta^2}{1+\beta^2}\exp\left(-\frac{2\sigma t}{\beta}\right) - \frac{1}{1+\beta^2}\left[\cos(2\sigma t) + \beta\sin(2\sigma t)\right]\right\}$$ **Equation 12**

The above equation assumes an idealised storm surge hydrograph based on the following function.

$$f(t) = \sin^2(\sigma t),$$ for $0<t<T_D$ **Equation 13**

Post storm, using the general exponential shoreline response equation (shown below) and the above calculated rate constant T_s, model results show a recovery period which responds considerably slower that the above calculate erosion rate. Kriebel and Dean (1993) note this is due to the fact that the beach profile is not as far out of equilibrium at the end of the storm when the water level drops as it is during the peak of the storm when the water level is elevated.

$$\frac{dR(t)}{dt} = \frac{1}{T_s}\left[R_\infty - R(t)\right]$$ **Equation 14**

Conceptually this response is realistic, however Kriebel and Dean (1993) note that during the beach recovery (post storm) smaller waves would likely impose a new rate parameter $T_{s,a}$ on the system that would further slow beach recovery. Furthermore, assuming the accretion R_∞ is equal to the pre-storm shoreline position is a major generalisation which will have significant impacts of following storm erosion calculations if multiple storms are modelled. During beach recovery, the maximum seaward position of the shoreline is strongly dependent on the wave height and water levels experienced post storm. For example, beaches with milder wave climates will respond significantly different to a location with a more active ambient wave climate. Similarly, seasonal variations in wave climate will influence the beach response for a particular location. As such the value for accretion R_∞ is not a constant value. It is dynamic, based on the ambient wave and water level climate experienced during the beach recovery period. The Kriebel and Dean convolution model does not account for this possible variation in accretion R_∞. Due to these two factors Kriebel and Dean (1993) do not recommend the use of the convolution method for estimates of beach recovery.

In addition to the issues regarding accretion modelling, assumptions neglecting the impact of tidal variations during storm events may reduce the accuracy of the Kriebel and Dean convolution method. The defined water level used in the Kriebel and Dean convolution method only represents the tidal anomaly. It does not account for the timing of events relative to tidal levels.

Recent work by Callaghan *et al.* (2008) focussing on the statistical simulation of wave climate and long-term beach erosion have applied the Kriebel and Dean convolution method for multiple storm events over long time scales in a monte carlo simulation approach to estimate longterm probabilities for shoreline erosion. The work by Callaghan *et al.* used a constant accretion R_∞ and an accretive rate parameter $T_{s,a}$ of 400h based on estimates of measurements from Ranasinghe *et al.* (2004b). Callaghan *et al.* (2008) does comment that the application of Kriebel and Dean (1993) to determine beach erosion is one aspect of their work which presents a clear limitation. For the Callaghan work this was acceptable, as the major focus of the paper was aimed at documenting statistical procedures to model wave climate and shoreline erosion. The use of Kriebel and Dean (1993) was only used as a test case to illustrate the use of the statistical procedures.

Testing of the Kriebel and Dean convolution method has been undertaken using survey data from the Gold Coast. The tests have found the constant accretion R_∞ assumption to be inappropriate and the accretive rate parameter ($T_{s,a}$) of 400h to be too low. These comparisons are documented in Section 5.2.

Miller and Dean

Miller and Dean proposed a model in 2004 which represented a significant improvement from the Kriebel and Dean convoltion model. The model proposed by Miller and Dean uses a time stepping finite difference approach and has the following features;

1. Independent rate parameters for erosion and accretion are defined. Enabling the model to be applied for both erosion and accretion modelling
2. Varying R_∞ for erosion and accretion are calculated for each timestep defined by instantaneous values of wave height and water level (Surge +Tide). This approach solves the problems with the Kriebel and Dean convolution model during erosion events by accounting for the current tidal level during storm events. Furthermore, this approach uses a dynamic function to define the accretion R_∞ value, instead of using the fixed value as was done by Callaghan *et al.* (2008).

These two factors overcome the major limitations of the Kriebel and Dean convolution approach.

The Miller and Dean model is based on the $Ax^{2/3}$ profile and continues using assumption, proposed by the Kriebel and Dean (1985) that shoreline response occurs at an exponential rate.

As mentioned above, instead of using the convolution method, Miller and Dean use a finite difference approach known as the Crank Nicholson scheme. Using this approach the linear response function can be represented by:

$$y^{n+1} = \frac{y^n + A\left[y_{eq}^{n+1} + y_{eq}^n - y^n\right]}{1 + \frac{k\Delta t}{2}} \qquad \textbf{Equation 15}$$

Miller and Dean (2004) note that the benefits of this approach are the computational efficiency of the finite difference scheme and the oscillatory nature of the forcing function which limits the build up of numerical error, as error tends to cancel rather than perpetually increase.

The Miller and Dean model is applied by using an equation developed by Miller and Dean to predict the "maximum" equilibrium profile shoreline position. This value, coupled with various parameters taken from the previous timestep is input into the Crank Nicholson scheme to predict the new shoreline position. The equation proposed by Miller and Dean to calculate the shoreline position is based on the $Ax^{2/3}$ profile (Bruun, 1954) and a Bruun-type conservation of volume argument. Driven by a combination of wave induced setup and storm surge the shoreline position equation is represented by:

$$y_{eq}^{n}(t) = y_0 + W*(t)\left(\frac{0.106H_b(t)+S}{B+2.0H_b}\right)$$ **Equation 16**

In order to test the robustness of the Miller and Dean model, 10 different sites were chosen, representing a variety of different coastal environments to calibrate and evaluate the model. To minimise anthropogenic effects, only sites located a sufficient distance from natural or manmade structures including jetties, groins, headlands, piers, and beach nourishment were selected. Initial model results were very promising, though uncertainty regarding the correct values representing the erosion and accretion rate parameters may have influenced the results at some of the localities (Miller and Dean, 2004).

An additional study by Miller and Dean, focusing directly on correct rate constant selection, was completed in 2006. The study used 13 sites, 10 within the USA and 3 within Australia. In total 225 simulations representing all possible combinations of k_a and k_e calculation methodologies were tested for each site. Figure 3-26 and Figure 3-27 show example plots of the results of the model validation tests for two of the thirteen locations (Miller and Dean, 2006).

The model verification process identified that regional groupings of the erosion and accretion rate constants produced the best results with the smallest normalised mean squared error. Figure 3-28 graphically shows the spread of accretion rate parameter values for four of the regional groupings.

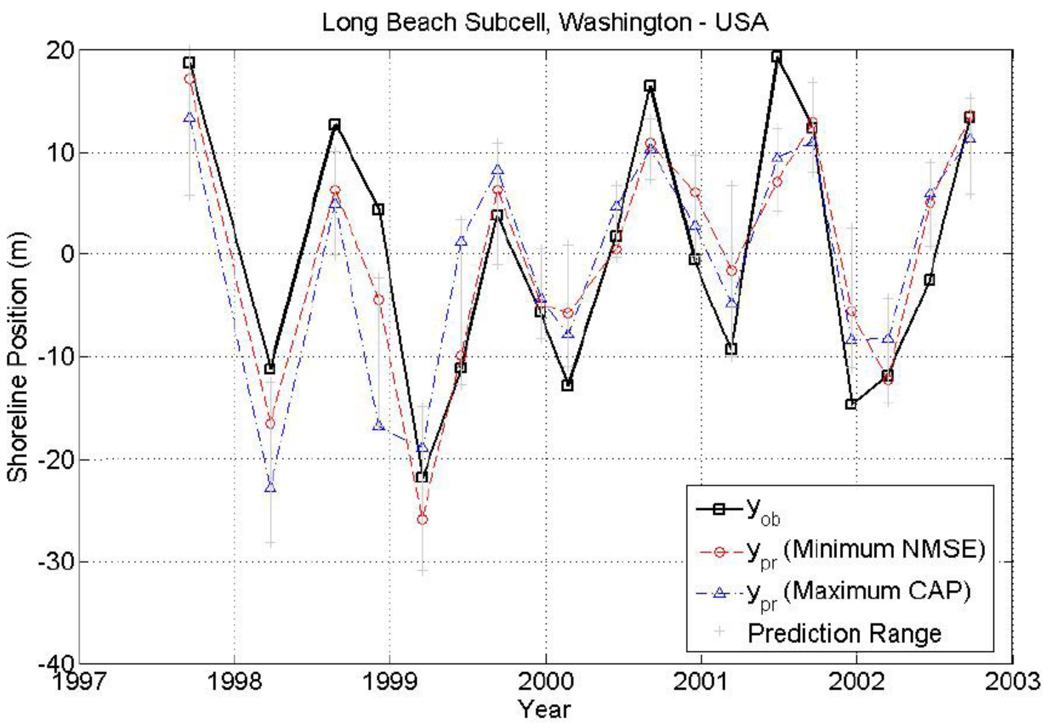


Figure 3-26 Miller & Dean Model Calibration – Long Beach (Miller & Dean, 2006)

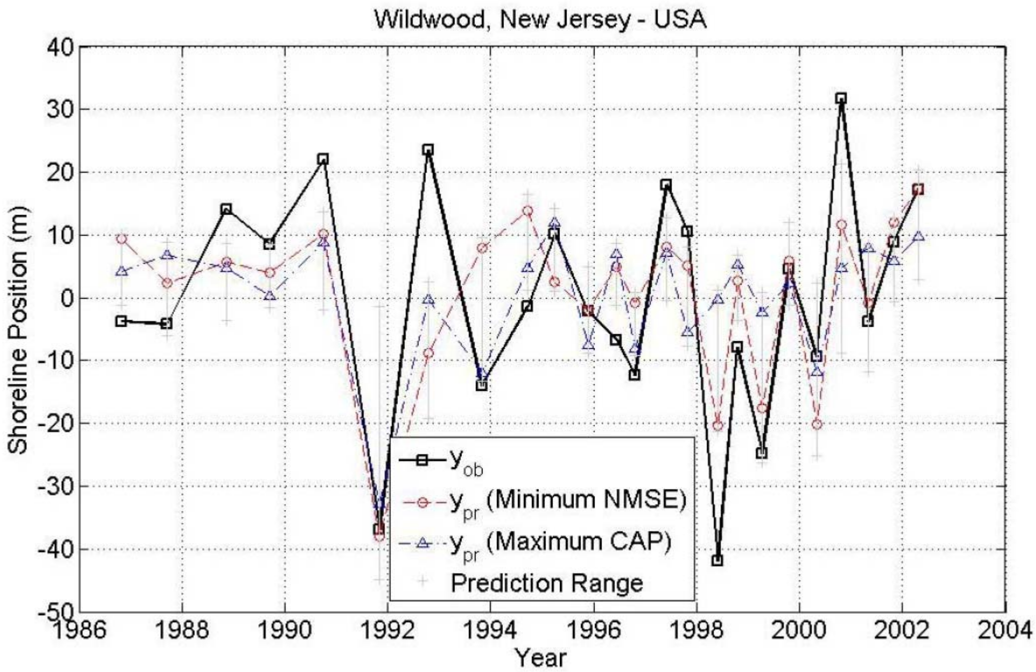


Figure 3-27 Miller & Dean Model Calibration – Wildwood (Miller & Dean, 2006)

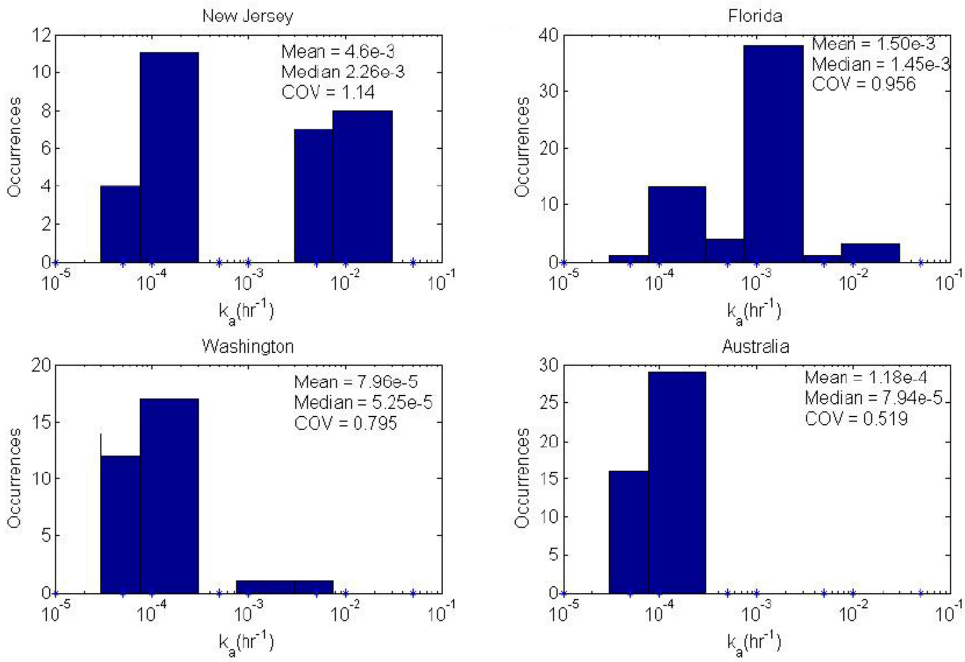


Figure 3-28 Accretion Rate Parameter Groupings (Miller & Dean, 2006)

Overall the results of the latest validation of the Miller and Dean model are promising, especially for modelling short-term (eg. successive storm/accretion events) events over long time frames (over years to decades), which require the prediction of shoreline accretion and erosion tracked over time. Furthermore, the basis of the model on the Bruun style conservation of mass principle make the model also applicable for the modelling of long-term responses driven by rises in sea level.

Although the Miller and Dean research validating their model shows promising results, there is concern that the application of the following formula may be an over simplification of the Bruun style mass conservation approach for sea level rise scenarios.

Figure 3-29 illustrates the application of the Equation 20 for an erosion scenario. Dean and Dalrymple (2002, p200) discuss the derivation of Equation 20, and illustrate a beach recession due to waves and tides, including wave set-up. Based on Dean and Dalrymple's discussions, it is evident that this equation only represents the cross-shore profile within the active surfzone width (*W**). The profile equation does not extend to the depth of closure. During climate change assessments, mass conservation out to the depth of closure is required. To meet these requirements as part of this Masters study, the model approach proposed by Miller and Dean (2004) has been modified. The modified approach uses the time stepping approach proposed by Miller and Dean. However, a geometric function extending from the dune beyond the depth of closure has been developed to replace Equation 20.

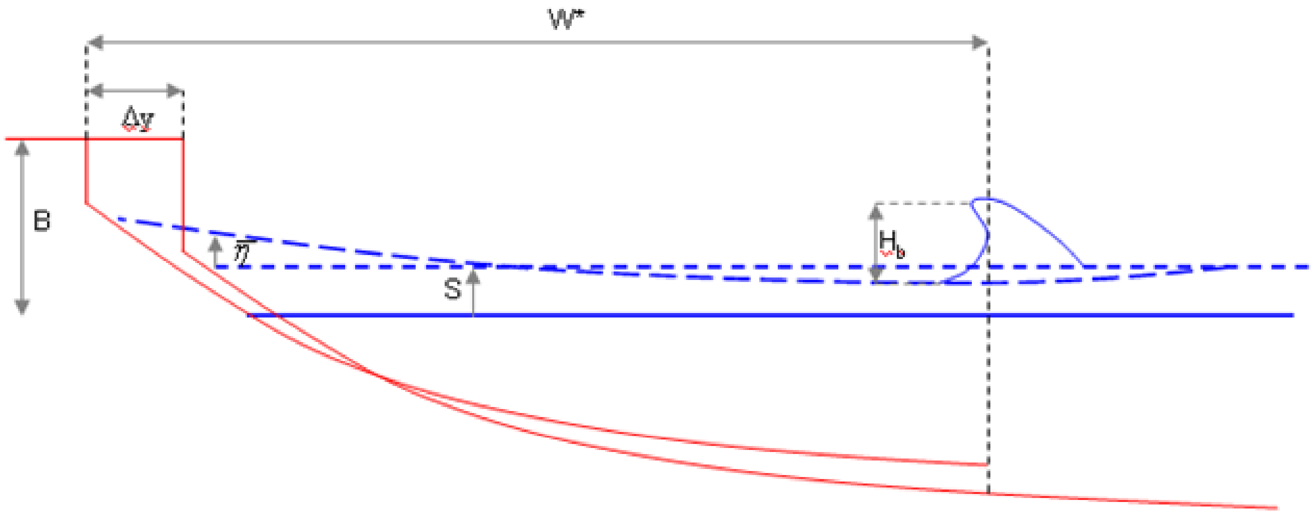


Figure 3-29 Beach Recession Due to Waves and Tides, Including Wave Set-up (Dean and Dalrymple, 2002)

3.3.4 Research Gaps

Due to the complexities associated with coastal engineering problems, assessments of cross-shore and longshore sediment are typically treated separately. Historically, climate change assessments have been based on cross-shore transport calculations using the Bruun Rule, often resulting in a single shoreline recession estimate for an entire beach length (assuming common berm height and depth of closure parameters). The impact of this cross-shore response on longshore sediment transport patterns has been neglected. Conversely, one-line shoreline evolution models are not capable of representing this shoreline response, as they do not account for cross-shore processes.

Based on results from this assessment, for littoral drift dominated coastlines, the interaction of these sediment transport processes is very important, resulting in the general shoreline response pattern described in Figure 3-30, not the linear recession estimate by the Brunn Rule. This research project aims to address this research gap by developing an assessment approach which jointly considers shoreline evolution accounting for combined cross/longshore processes.

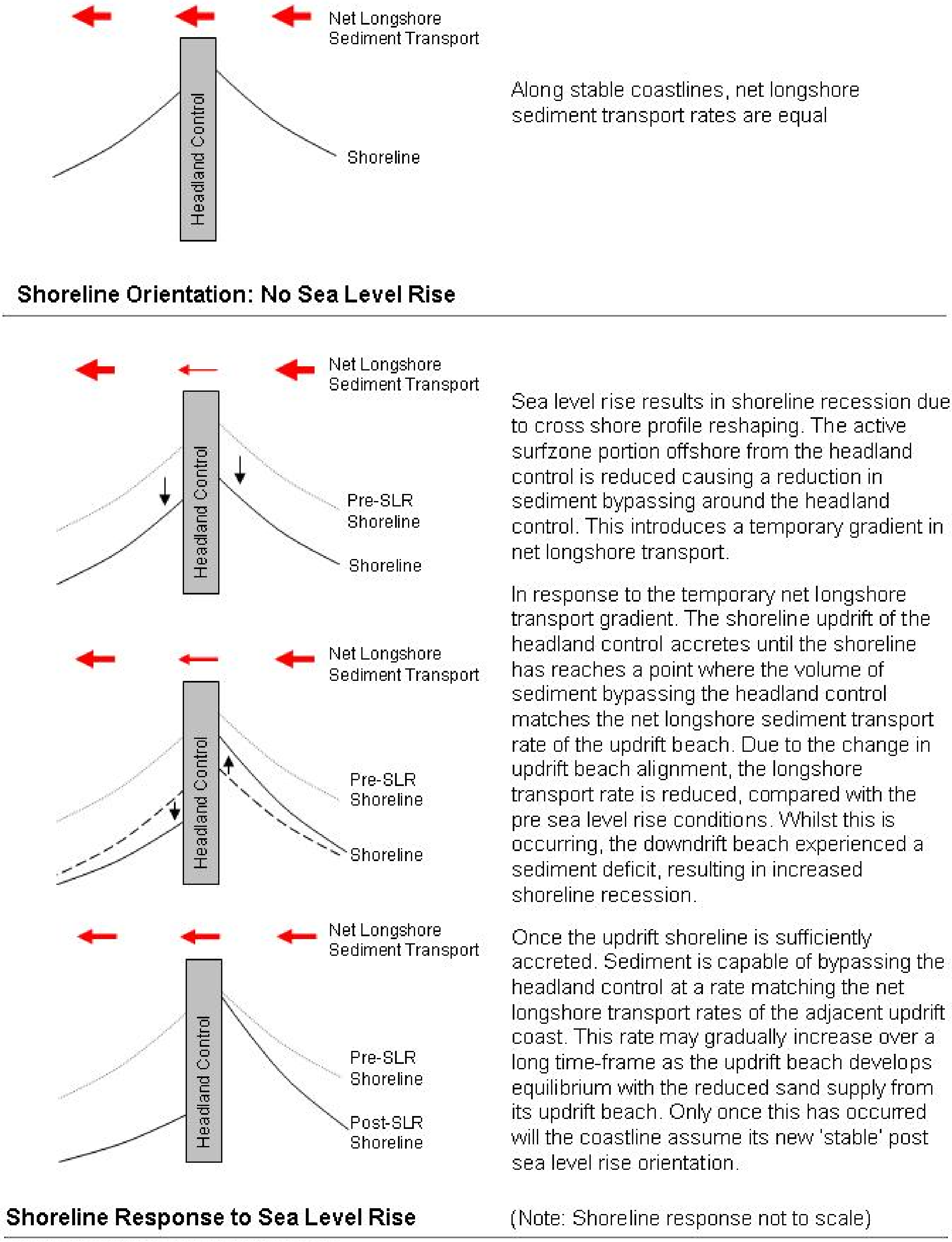


Figure 3-30 Cross/Longshore Shoreline Response to Sea Level Rise

4 MODEL DEVELOPMENT

4.1 Model Selection

Due to the long simulation times required for climate change modelling to the 2100 planning horizon, development of a modelling approach which is both computationally efficient and also dynamically stable is required.

Hanson *et al* (2003) summarises the challenges associated with fine scale process modelling based on first principles regarding long-term shoreline evolution modelling.

> *"At present, knowledge is very limited on the motion of sediment particles in spatially varying flows of a combination of mean and oscillatory currents, especially together with turbulence induced by the breaking waves. Numerous other complicating factors, such as the complex fluid motion over an irregular bottom and absence of rigorous descriptions of broken waves and sediment-sediment interaction, also make the problem of computing sediment transport and associated beach change essentially impossible if a first-principles approach at the micro-scale is taken".*

Due to these limitations, shoreline evolution modelling based on dynamic equilibrium principles are traditionally seen as the preferred long term modelling approach. These models are typically less computational intensive and additionally tend to be less exposed to the possibility of cumulative error adversely affecting the model results.

Due to these factors, an equilibrium cross-shore and one-line longshore model has been developed. Both of these model types have been shown to be capable of modelling long timeframes (decadal) with satisfactory results (Hanson *et al*, 2003). Although one-line models are not truly equilibrium based models, assessment of the dynamic stability of the model simulations over long timeframes is commonly used to verify that one-line modelling is 'stable'. This stability verification was successfully completed during this study, prior to and during the 100 year long simulations undertaken as part of the climate change assessment for Wooli Wooli Beach.

The longshore and cross-shore models developed as part of this study are described in the following sections.

4.2 Longshore Modelling – LSMOD

One-line modelling has traditionally been used to assess shoreline response to proposed engineering works in coastal areas where longshore sediment transport represents the dominant shoreline evolution forcing. GENESIS, developed by Hanson and Kraus (1989), represents a commonly used commercially available one-line model. Currently, GENESIS cannot assess the impact of climate change related sea level rise on shoreline response. The modelling package does not represent the cross-shore sediment transport processes impacted by rising sea levels, and the interaction these cross shore sediment transport changes have on longshore sediment transport gradients.

This limitation has been addressed within this study by developing both longshore and cross-shore sediment transport models which have subsequently been dynamically linked. Outlined below is a description of the developed one-line longshore model, titled LSMOD.

LSMOD calculates longshore sediment transport driven changes in shoreline position based on input sediment, profile and deepwater wave conditions. The model uses the CERC equation in a fixed grid framework to model shoreline change resulting from gradients in longshore sediment transport.

A schematic representing the general model features is shown in Figure 4-1. The one-line model uses a single shoreline contour to represent the coastline. By calculating alongshore fluxes in wave energy, shoreline change driven by gradients in longshore sediment transport are calculated by the model.

The developed model uses deepwater wave conditions, sediment size data, active profile limits and initial shoreline orientation information as input details. Internal to the model, in order to apply the CERC formula within a fixed grid representation, two main additional time varying parameters are calculated for each timestep of the model simulation. These parameters are:

- Breaking wave conditions; and
- Shoreline orientation details.

Breaking wave conditions are obtained via a two step approach. Initially, wave transformations tables are required to translate nearshore wave conditions given deepwater inputs. From the nearshore nest locations, linear wave transformation is used to calculate the breaking wave conditions for each cell side within the model.

This two step approach wave transformation procedure was favoured over parallel contour assumptions use by other one-line models as it provides increased accuracy in the representation of nearshore wave input data. In locations such as Wooli Wooli, where offshore islands and reefs influence changes in swell energy and incoming wave ray angles, correct representation of the offshore wave transformation is vital to obtain accurate longshore sediment transport estimates.

To transform the deepwater wave input to the nearshore, the modeller is required to define the nearshore wave nesting location. It is recommend that the nesting locations be located offshore in approximately 15m of water depth with spacing between points of approximately 5 to 10 grid cells. For all grid cells between the defined nest locations, wave conditions are linear interpolated. An example model domain, showing various wave nest locations is shown in Figure 4-1. For each of these nesting locations, specific wave transformation tables are required. Wave transformation tables represent a matrix of conversion factors relating nearshore wave heights and directions to deepwater inputs of wave height, direction and period. An example wave direction transformation table is shown by Table 4-1. To populate these transformation tables, various wave models may be applied, though for this assessment the wave spectra model SWAN has been used.

Table 4-1 Example Wave Direction Transformation Table

Deep Water Wave Direction (Shore Normal)	Deep Water Wave Period (Tp) 2	4.5	7	9.5	12	14.5	17	19.5
90	-62	-70	-79	-86	-90	-92	-94	-96
67.5	-43	-51	-60	-66	-69	-72	-74	-75
45	-29	-37	-44	-49	-52	-54	-56	-58
22.5	-17	-24	-30	-34	-36	-38	-40	-41
0	-8	-13	-17	-19	-21	-23	-24	-25
-22.5	-1	-4	-5	-6	-6	-7	-7	-8
-45	6	6	7	8	9	9	10	10
-67.5	15	17	20	23	25	27	28	29
-90	30	32	36	41	44	46	48	49

Note: Nearshore wave direction calculated by adding transformation value to deepwater wave direction

Once the breaking wave conditions are obtained, the shoreline orientation for each cell side is calculated. The shoreline orientation is calculated based on the inverse tangent of the difference between the neighbouring cell centres divided by the model grid cell width. This is represented in Equation 17.

$$\alpha_s = a\tan\left[\frac{(xc_{n-1} - xc_n)}{dx}\right]$$

Equation 17

Using the cell side shoreline angle, the relative angle of the breaking wave condition is calculated. Based on this angle and the previously transformed breaking wave height, the sediment transport volume for each cell side is calculated using the CERC formula (Equation 1).

To translate the calculated sediment transport volumes to a shoreline change estimate, the difference between each cell side sediment transport volume is calculated and divided by the active surfzone height and the model grid cell width (Equation 18). These model features are represented in Equation 18 and displayed in Figure 4-2.

$$dy = \frac{dt(Q_{n-1} - Q_n)}{dx(D_b + D_c)}$$

Equation 18

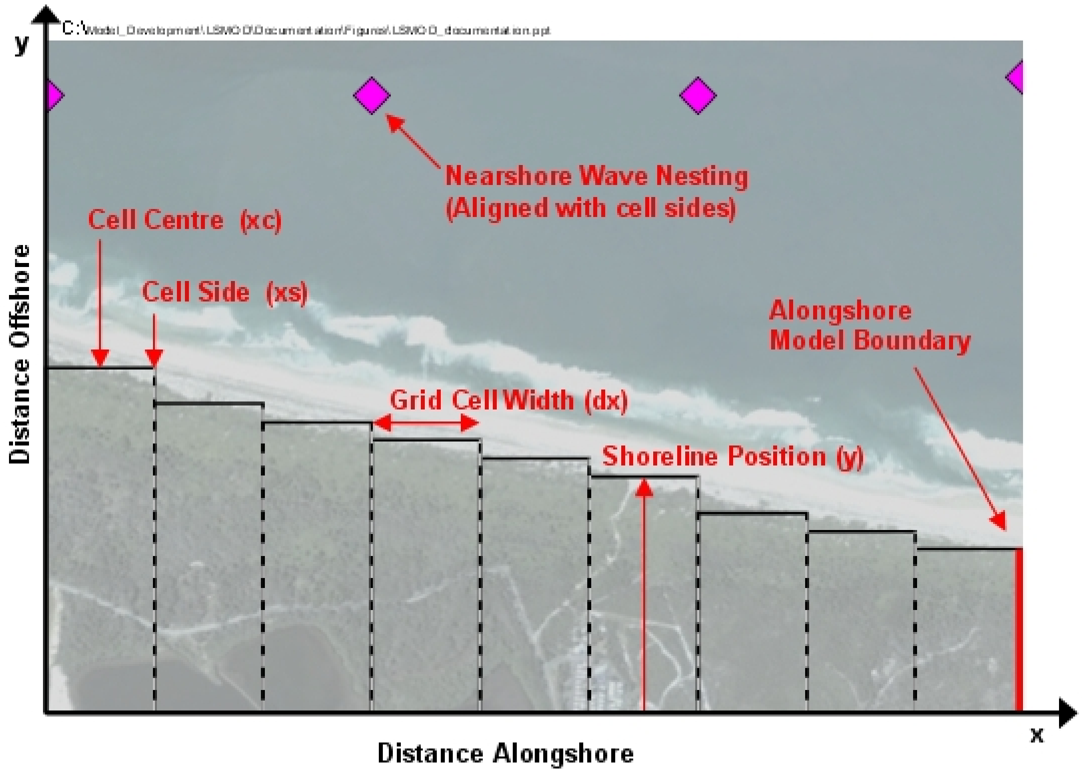


Figure 4-1 LSMOD Model Layout

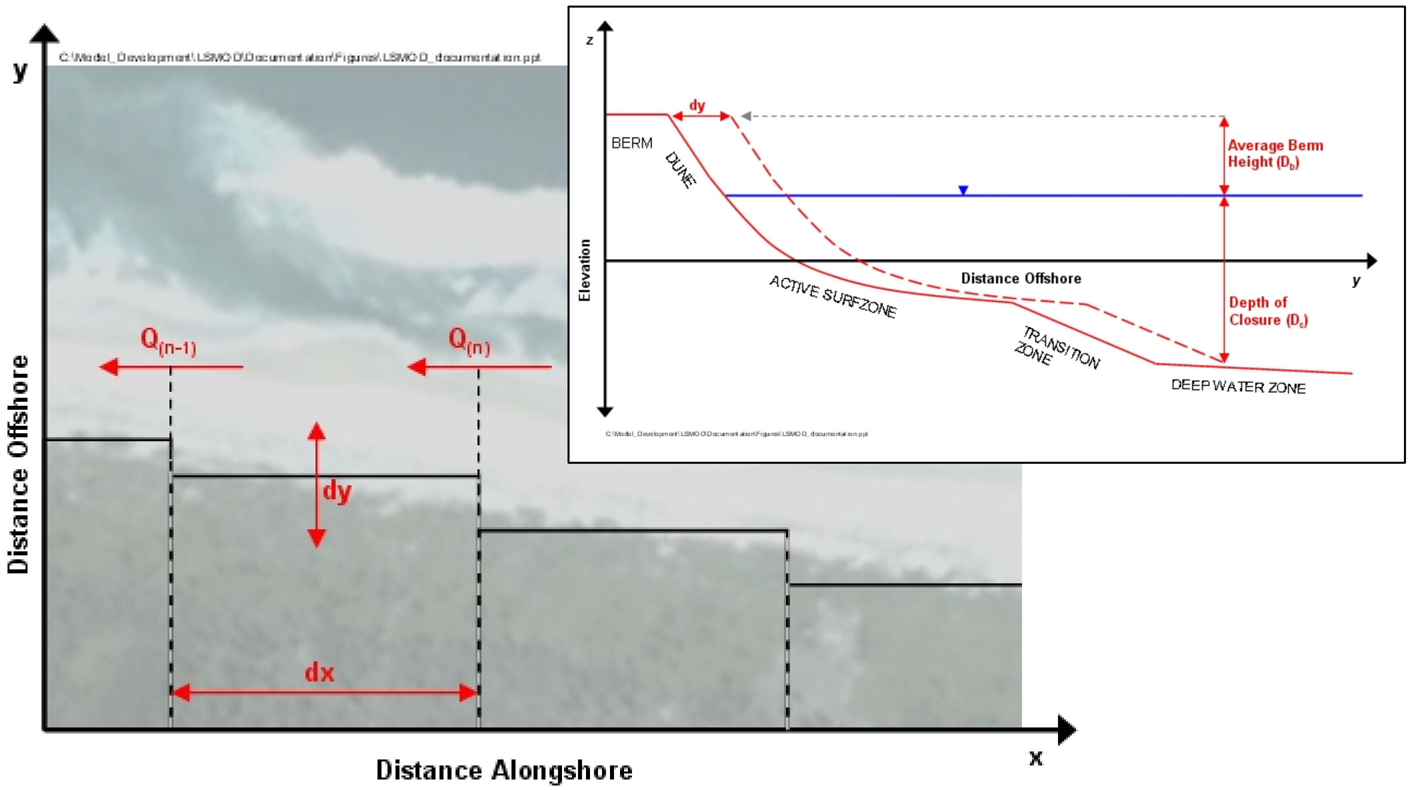


Figure 4-2 LSMOD Shoreline Change Representation

The steps listed above represent the basic model setup. In addition to this, various alongshore model boundary condition types have been included. The boundaries included:

1. Pinned Boundary – Fixes the alongshore boundary cell to a defined location for the entire simulation. This boundary type assumes there is no gradient in longshore sediment transport at the model boundary;

2. Open Boundary – The alongshore boundary is free to move dependant on wave energy fluxes calculated at the model boundary. These fluxes are calculated based on the gradient of the sediment transport gradient experienced by adjacent updrift/downdrift cells. Where a gradient in transport gradient exists, the trend in gradient is extrapolated to the model boundary;

3. Periodic Boundary – This boundary approach is only appropriate for use along coasts with dominant littoral drift trends exhibiting unidirectional sediment bypassing of headland controls. This boundary type has been developed requiring the representation of headland/groyne features coinciding with both boundary cells in the model. When sediment is calculated to bypass the given downdrift headland, equal sediment volume is input to the opposite, updrift model boundary.

In addition to the above listed boundary types, groyne or headland control features have also been included in the developed model. The volume of sediment bypassing the modelled groyne/headland is calculated by identifying the proportion of the total potential updrift longshore sediment transport corresponding to the active surfzone width offshore from the groyne/headland tip.

Existing one-line models are not able to assess the impacts of climate change. Shoreline recession related to sea level rise is a cross-shore sediment process which longshore models can not account for. In the absence of any accepted 3-D models suitable for long-term simulations (>50 year), required for a climate change assessment, this issue has been resolved by dynamically linking LSMOD with a developed cross-shore model (XSMOD) (discussed in Section 4.4). The dynamic linking of the longshore and cross-shore models uses a link frequency of 30 days. Although it would be beneficial to use a more frequent linking interval, extended simulation times limited the minimum acceptable value for the link frequency. The developed cross-shore model and dynamic linking is discussed in following sections.

Overall, the model structure and features used by LSMOD is not a new concept. GENESIS (Hanson and Kraus, 1989), which has been commercially available for the last two decades, uses a similar approach. Rather than using GENESIS as the longshore model, however, to facilitate the dynamic linking of the longshore model to the developed cross-shore model (XSMOD), LSMOD was developed. Development of a new longshore model as part of this study, compared with using an existing software package, offered the following benefits:

1. Input file format control (The boundary condition and wave transformation input data used by XSMOD and LSMOD use consistent formatting);

2. Increased result output and internal reporting capabilities (GENESIS, offers limited output and model check file options); and

3. Flexibility during the development of the longshore/cross-shore model dynamic linking procedure.

4.3 Cross-shore Modelling – XSMOD

A time stepping cross-shore model, titled XSMOD, has been developed as part of this Masters study. The model is based on a cross-shore modelling approach described by Miller and Dean (2004), further developing the cross-shore profile representation in the model to be suitable for climate change assessments.

4.3.1 Profile Survey Data

To create the geometric function to replace Equation 16 in the Miller and Dean (2004) cross-shore model, survey data from the Gold Coast in South Eastern Queensland has been used. In total, 40 cross-shore profile surveys extending from the upper dune to the depth of closure are available for the Gold Coast. The survey information provides accurate profile information from 1966 through till 2002.

Plotting of the available survey datasets enabled the creation of the geometric function extending the $Ax^{2/3}$ profile offshore from the active surf-zone beyond the depth of closure. Figure 4-3 and Figure 4-4 show some examples of the available Gold Coast survey data.

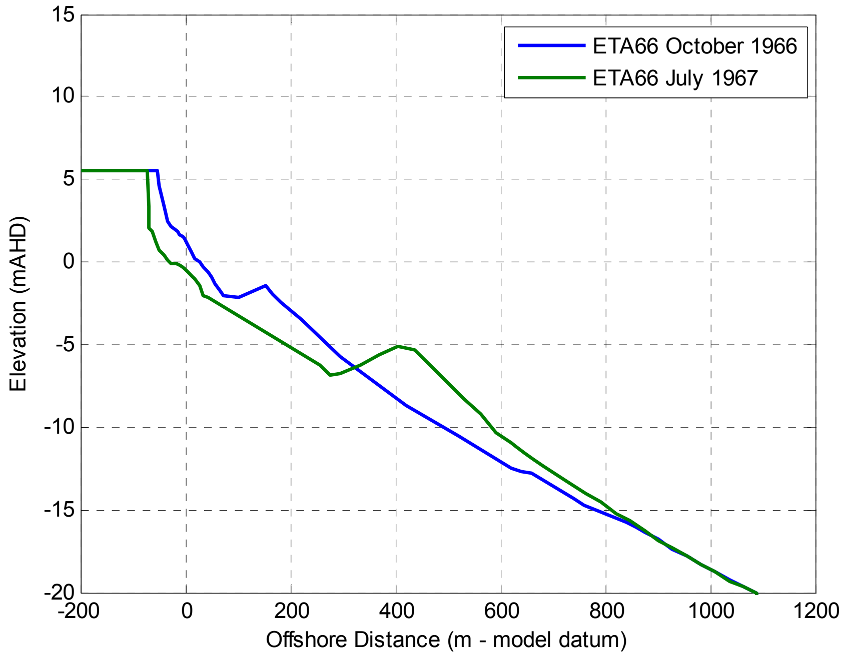


Figure 4-3 Gold Coast ETA 66 Cross-Shore Survey

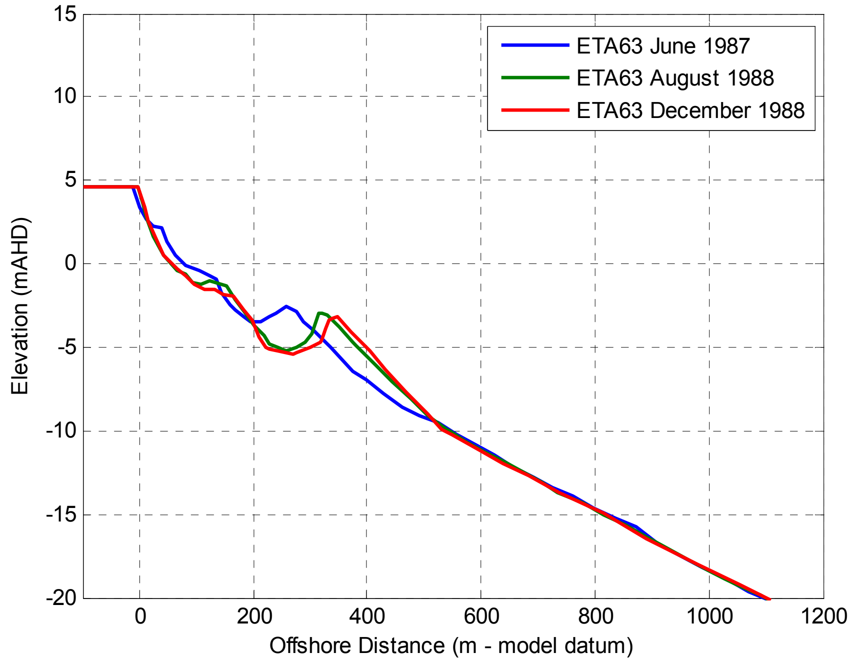


Figure 4-4 Gold Coast ETA 63 Cross-Shore Survey

Analysis of the Gold Coast survey data identified the occurrence of five common sections to cross shore profile. These include the Berm, Dune, Active Surf Zone, Transition Zone; and Deepwater Zone. Figure 4-5 identifies these sections of the cross-shore profile.

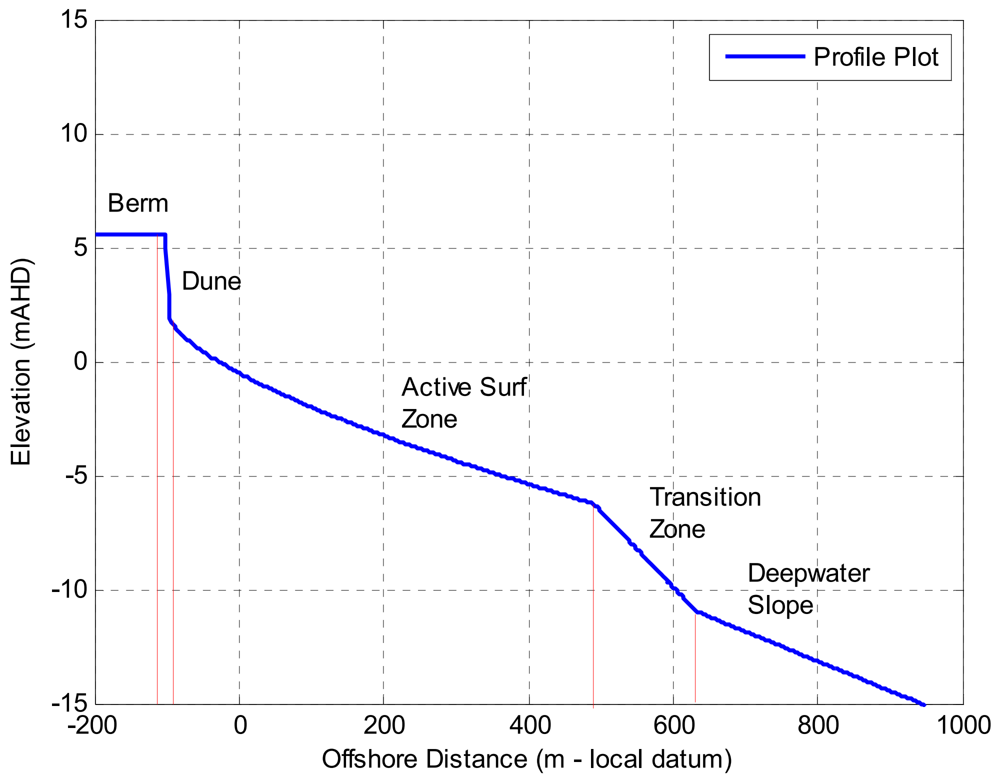


Figure 4-5 Profile Breakup

Offshore from the active surf-zone two separate features were identified; the transition slope and a deepwater slope. It is possible that:

1. The transition slope is most probably grainsize dependant.
2. Beyond the depth of closure the constant deepwater slope was most probably developed over geologic timescales during the past sea level rise occurrence.

Further research is required to verify these assumptions.

Based on the Gold Coast survey data, the deepwater slope was consistently found to be approximately 1:60 (Vertical:Horizontal) and fixed at all times, being seaward of the depth of closure. Due to the deepwater sections constant slope and position, geometrically the slope can be represented by the general equation for a straight line where *b* represents a fixed value at the intersection of the defined y-axis.

$$y = mx + b \qquad \textbf{Equation 19}$$

Shoreward of the deepwater slope the transition slope was found to be approximately 1:30 (Vertical:Horizontal). Dependent on the width of the active surf zone the transition slope shifts along the deepwater slope whilst maintaining the above mentioned 1:30 slope. Similar to the deepwater slope, the transition slope can be represented by the general equation for a linear function (Equation 22). However, due to the shift in location of the transition slope, dependant on the active surfzone width, the value for *b* varies.

By including these two equations with the linear equations representing the berm, dune and non-linear $Ax^{2/3}$ representation of the active surf zone, an $Ax^{2/3}$ profile can be represented from the dune beyond the depth of closure.

4.3.2 Model Construction

The cross-shore modelling approach adopted by Miller and Dean assumes that the shoreline responds according to the linear response model given by:

$$\frac{dy}{dt} = k(y_{eq} - y) \qquad \textbf{Equation 20}$$

Where $y_{eq}(t)$ represents the instantaneous position of the equilibrium shoreline, $y(t)$ is the instantaneous position of the actual shoreline and k is a rate constant.

Miller and Dean adopted an empirical equation relating wave height, berm height, water level and active surf zone width to shoreline position, shown in Equation 21

$$\Delta y_{eq}(t) = -W^{*}(t)\left(\frac{0.106H_b(t) + S}{B + 2.0H_b(t)}\right) \qquad \textbf{Equation 21}$$

The implicit Crank Nicholson scheme (Equation 22) was used as it is unconditionally stable for k>1.

$$y^{n+1} = \frac{y^n + A[y_{eq}^{n+1} + y_{eq}^{n} - y^n]}{1 + A}, A = \frac{k\Delta t}{2} \qquad \textbf{Equation 22}$$

Miller and Dean successfully validated their model approach using surveyed shoreline information for 13 study areas. The model approach was simulated for varied timescale durations (up to 40years), dependent upon the availability of survey data. The model validation identified the model approach to be suitable for predicting both short-term and long-term changes in shoreline position.

Assessment of the available time stepping cross-shore modelling approaches determined the Miller and Dean approach to be the most suitable method for the long-term simulations. There was concern, however, that Equation 21 developed by Miller and Dean may be an over-simplification of the cross-shore profile representation climate change modelling assessing the impact of long-term sea-level rise. To manage this concern, a geometric approach has been developed to replace Equation 21. The geometric profile based on observed offshore profiles from the Gold Coast on the east coast of Australia.

The developed geometric approach utilises four linear functions and one non-linear function to represent the “actual” cross-shore profile and the “equilibrium” recession/accretion profile for a given timestep. The finite difference approach given by Equation 22 is used to evolve the shoreline position over time using calculated shoreline position inputs, extracted from the actual and equilibrium profiles from any given timestep.

Using limiting functions for each expression, the following five equations are used to define the profile shape from the berm through to deep water, beyond the depth of closure. Figure 4-6 shows an example of a profile constructed using the following equations.

$$y = B$$ **Equation 23: Berm**

$$y = y_{d0} - m_0\left(x - \Delta x\right)$$ **Equation 24: Dune**

where: $$y_{d0} = -\left(\frac{-n_b - Kh_b}{1-K}\right)$$

$$y = \frac{-Ax^{\frac{2}{3}} - n_b - Kh_b}{1-K}$$ **Equation 25: Active Surf-zone**

$$y = \frac{1}{-m_1}x + b_1$$ **Equation 26: Transition Slope**

where: $$b_1 = y_b - \frac{1}{-m_1}x_b$$

where $$x_b = \left(\frac{(h_b - n_b)(1-K) + n_b + Kh_b}{A}\right)^{\frac{3}{2}} + \Delta x$$

where $$y_b = \frac{-A(x_b - \Delta x)^{\frac{2}{3}} - n_b - Kh_b}{1-K}$$

$$y = \frac{1}{-m_2}x + b_2$$ **Equation 27: Deepwater Slope**

To apply the profile information defined by Equation 23 to 27 using the finite difference time stepping approach the following procedures are followed.

1. The initial $Ax^{2/3}$ profile is calculated based on available survey, applying the conservation of mass principles discussed in Section 3.3.3.2. Figure 4-7 shows a $Ax^{2/3}$ profile compared with survey data from the Gold Coast, Australia.
2. The “equilibrium” or maximum response profile, based on the wave height and water level for the next timestep is plotted. The location of the profile is shifted with reference to offshore distance to ensure erosion/accretion of berm features is offset by an equal volume of accretion/erosion to the offshore bar. Figure 4-8 shows this process for a case where the water level and wave height both increase for the next timestep.
3. The future profile (n+1) is plotted using the calculated future shoreline position, derived using the Crank Nicholson scheme transcribed along the linear slope between the current (old) shoreline position and the equilibrium response position.
4. To conserve mass between the future profile (n+1) and the current profile (n) the location of the transition point from the $Ax^{2/3}$ profile to the transition slope is adjusted on/offshore. This is required due to the non-linearity of the $Ax^{2/3}$ profile (A shift in shoreline position between timesteps does not equate to an equal shift in the defined breaker position). Shifting this location represents a change in the profile shape resulting for a change in wave energy. This is shown in Figure 4-9.
5. The future profile (n+1) is then used to define the current profile (n) and current shoreline position (n) for the next timestep.
6. Repeat steps 2 through 6.

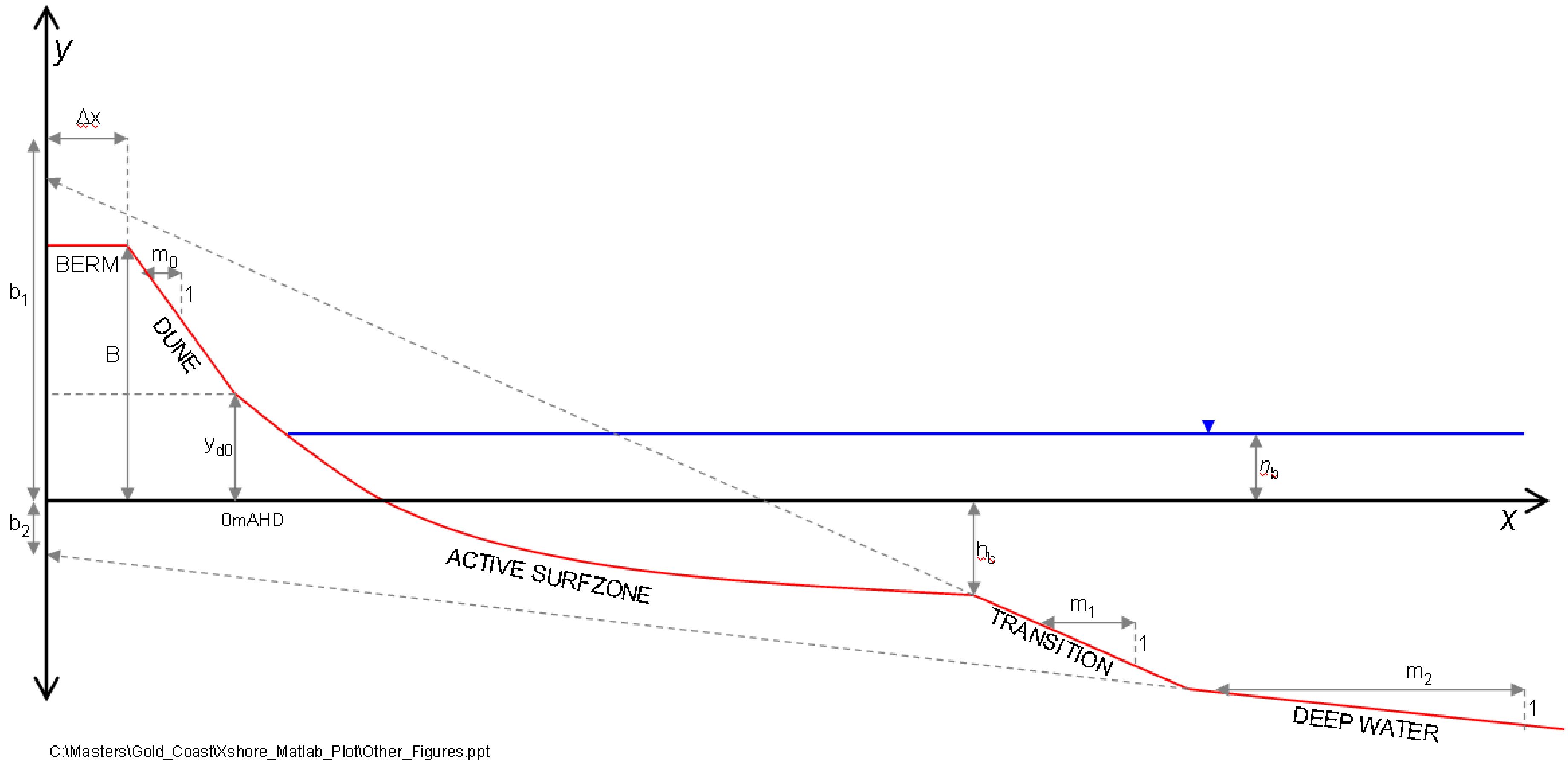


Figure 4-6 Cross-Shore Model Parameter Definition

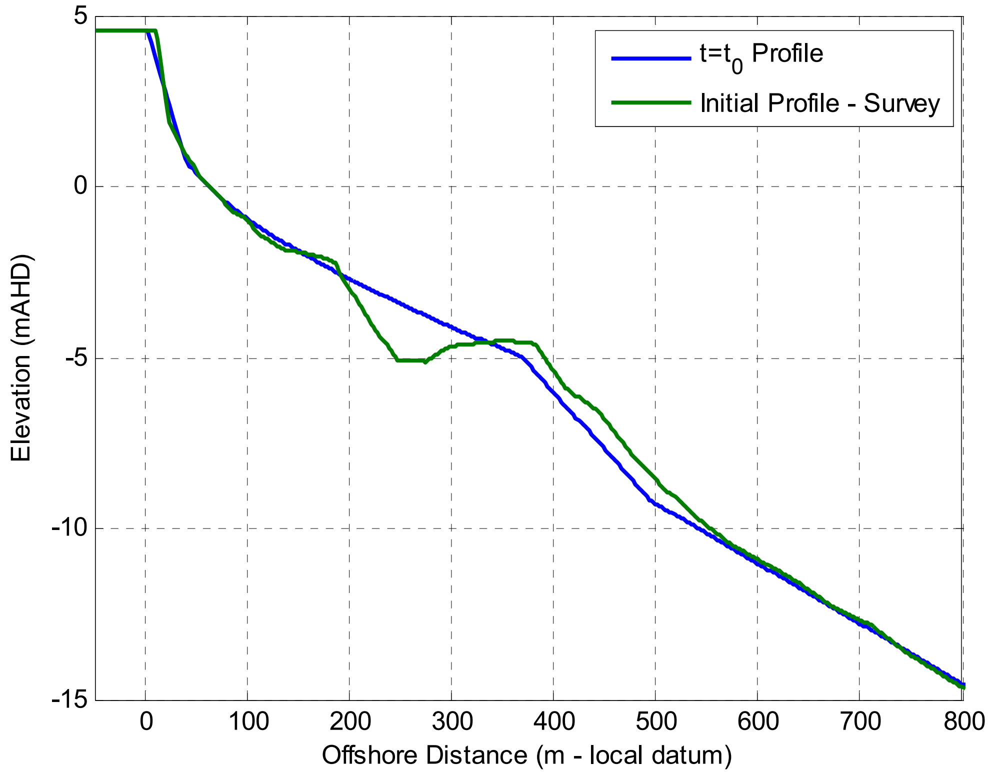


Figure 4-7 Representative $Ax^{2/3}$ Profile

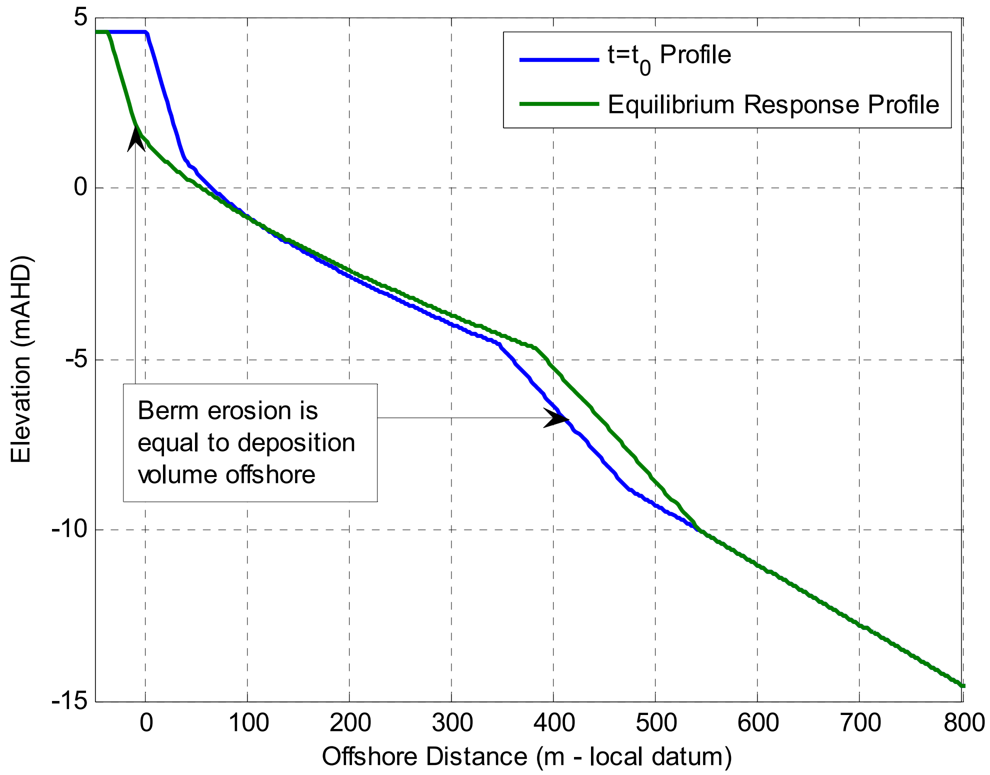


Figure 4-8 Equilibrium Profile Plot

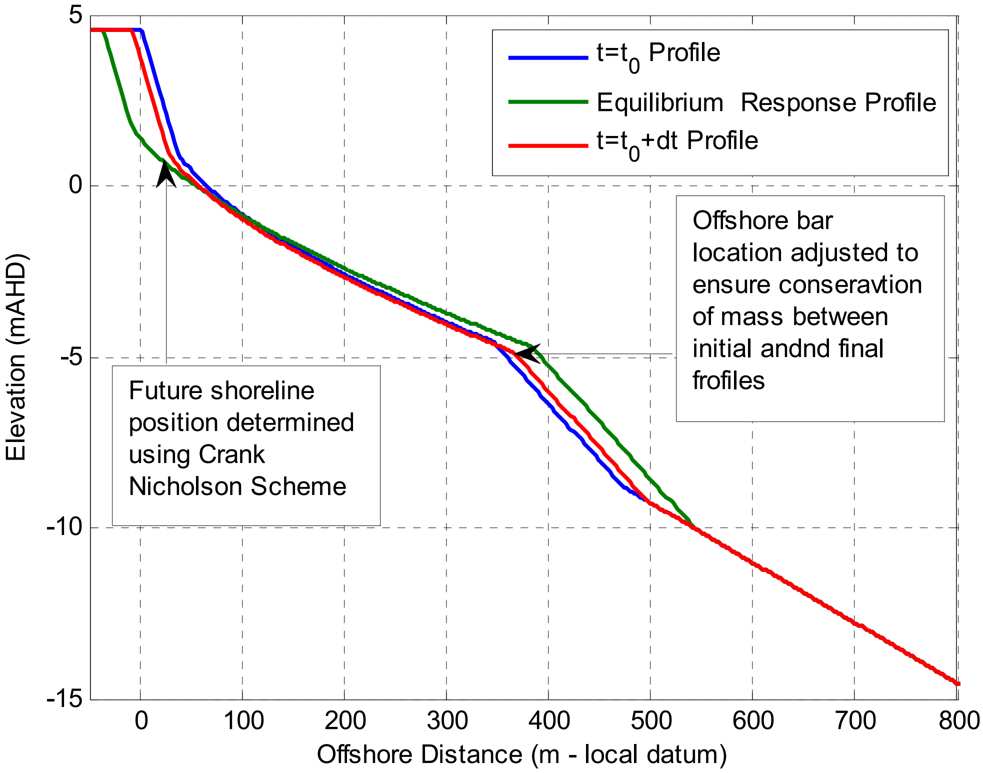


Figure 4-9 Future Profile Plot

Figure 4-10 and Figure 4-11 show the application of the XSMOD program for two extreme hypothetical test cases. Although the hypothetical cases represent highly unrealistic wave height/water level situations, they do suit the purpose of illustrating the model operation well.

Figure 4-10 represents a hypothetical situation where the wave height is indefinitely held constant. At the first timestep the water level is increased by 2.5 m, after which it remains constant at the elevated level for the remainder of the simulation. The results show how the elevated water level results in the prediction of an upward translation of the profile. Erosion is predicted from the upper section of the profile. The eroded material is deposited offshore, facilitating the upward shift of the profile.

Figure 4-11 represents a hypothetical situation where the water level is fixed whilst an increase in wave height is experienced. At the first timestep the significant wave height is increased to 6.5m, after which it remains constant at 6.5m for the remainder of the simulation. The results show how the increased wave height results in the lengthening of the active profile. Erosion is predicted from the upper section of the profile. The eroded material is deposited offshore extending the profile into deeper water, accommodating for the increase in wave energy.

For both hypothetic test cases the decreased spacing for between the plotted results for the later sample periods illustrate the application of the exponential shoreline response function used by the XSMOD model.

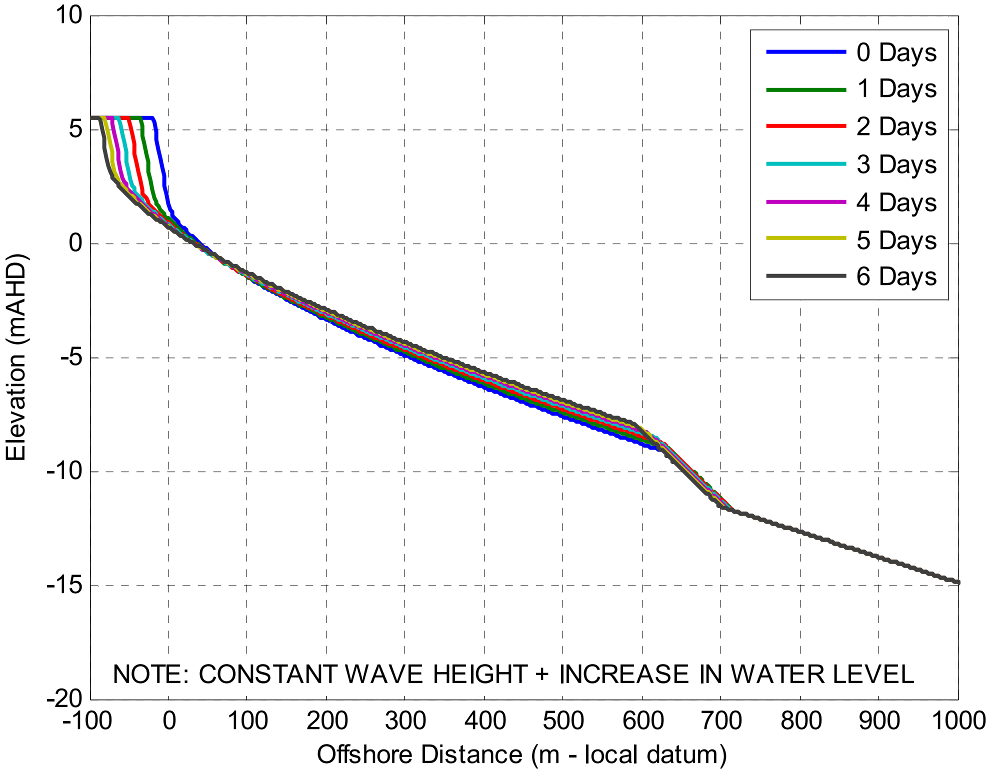


Figure 4-10 XSMOD Profile Change Example – Water Level Increase

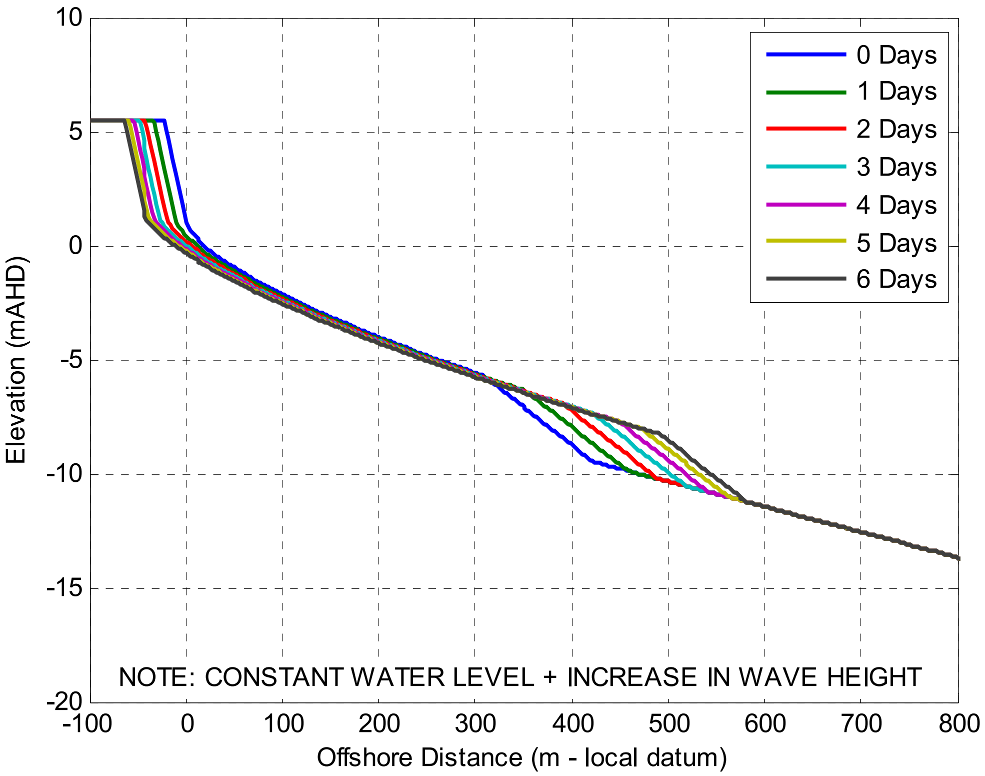


Figure 4-11 XSMOD Profile Change Example – Wave Height Increase

4.4 Combined Cross-Shore / Longshore Modelling

Assessment of the historic survey data for the case study location assessed during this study (Wooli Wooli Beach) indicates non-uniform alongshore erosion response to cross-shore sediment transport. This is predominantly due to wave diffraction and/or refraction resulting in alongshore variations in wave heights during storm events. Typically, more exposed sections or a beach length are likely to experience larger cross-shore storm erosion than more sheltered locations.

This non-uniform response to cross-shore driven changes in shoreline position is likely to impact upon post storm longshore transport gradients. Similarly, shifts in shoreline position resulting from longshore gradients are likely to have an impact on cross-shore erosion volumes.

To accommodate this cross-shore/longshore sediment transport interaction, the dynamic coupling of the cross-shore and longshore models is required.

A procedure has been developed enabling the dynamic coupling of the cross-shore and longshore models mentioned above. Using Matlab as the processing program, the coupling procedure uses a 30 day linking interval to dynamically link the cross-shore and longshore models.

The general steps required to link the two modelling programs is outline below and shown in Figure 4-12.

1. Following simulation of a given 30 day period, the final shoreline position calculated by both models, and the evolved cross-shore profile is extracted.

2. The variation in shoreline position with reference to the initial position for the 30 day simulation is summed. The summed change in shoreline position value represents the net shoreline change due to both sediment transport processes.

3. The calculated net shoreline change value is used to update the initial shoreline position for the following 30 day simulation.

4. The above steps are repeated until the desired simulation period is modelled.

The 30 day linking interval was chosen as it represented a linking interval which resulted in sufficiently accurate results without significantly impacting on simulation runtimes. During the model development stage various linking intervals ranging from weekly (7 days) to annually (364 days) were trialled. It was found that link periods from 60 to 365 days resulted in shorter simulation runtimes, but also in less accurate results. This was apparent, where increasing the link period resulted in differing simulation results. In comparison the simulations using a linking interval less than 30 days resulted in excessive runtimes for a negligible increase in model result accuracy (little variation in simulation result irrespective of the link interval reduction).

Throughout the model simulation the modelled shoreline position and the most landward shoreline position for the entire simulation are tracked. The most landward shoreline position is tracked to assist in the definition of hazard lines. Figure 4-12 outlines the general steps required to link the modelling programs.

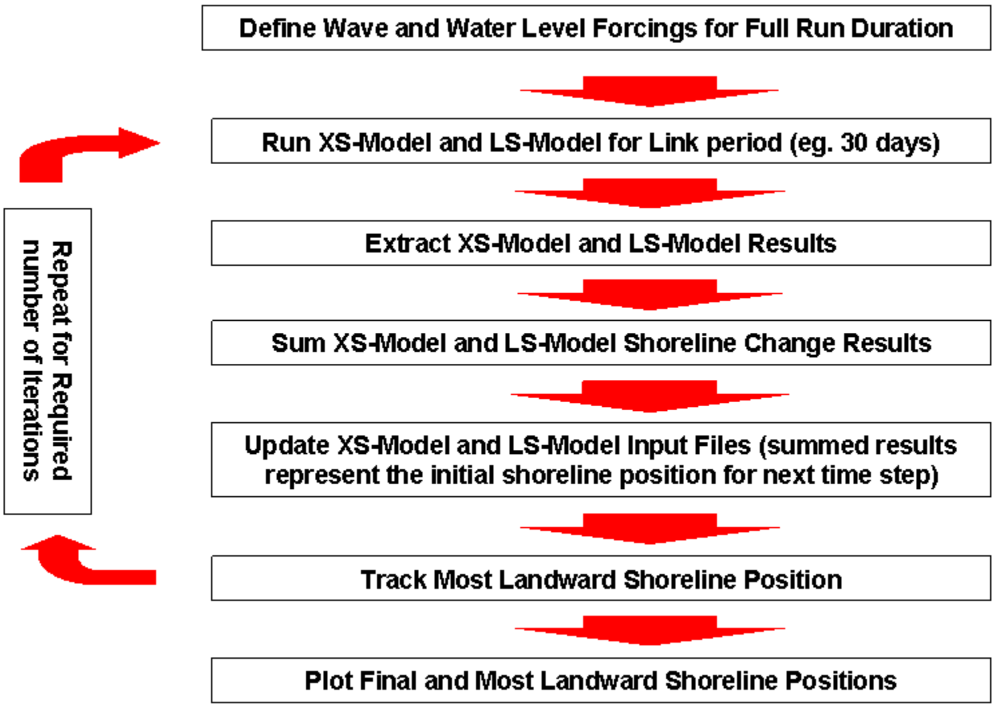


Figure 4-12 Cross-Shore/Longshore Model Coupling Procedure

5 MODEL VALIDATION

5.1 Gold Coast

The Gold Coast in south east Queensland represents one of the most documented stretches of coastline on the eastern seaboard of Australia. Extensive wave and coastal survey data has been collected along the Gold Coast since the late 60s.

In total, 40 cross-shore profile surveys extending from the upper dune to the depth of closure are available for the Gold Coast. The survey information provides accurate profile information from 1966 through till 2002.

Regarding wave data, non-directional wave data is available via the Brisbane wave rider buoy from 1976 to 1996. From 1996 to the present, directional wave data is available for the Brisbane buoy. Additional non-directional wave data is also available from the Gold Coast wave rider buoy from 1987 through to 2007. Following the completion of the initial model calibration, recorded directional data for a three month period in 2007 was also obtained.

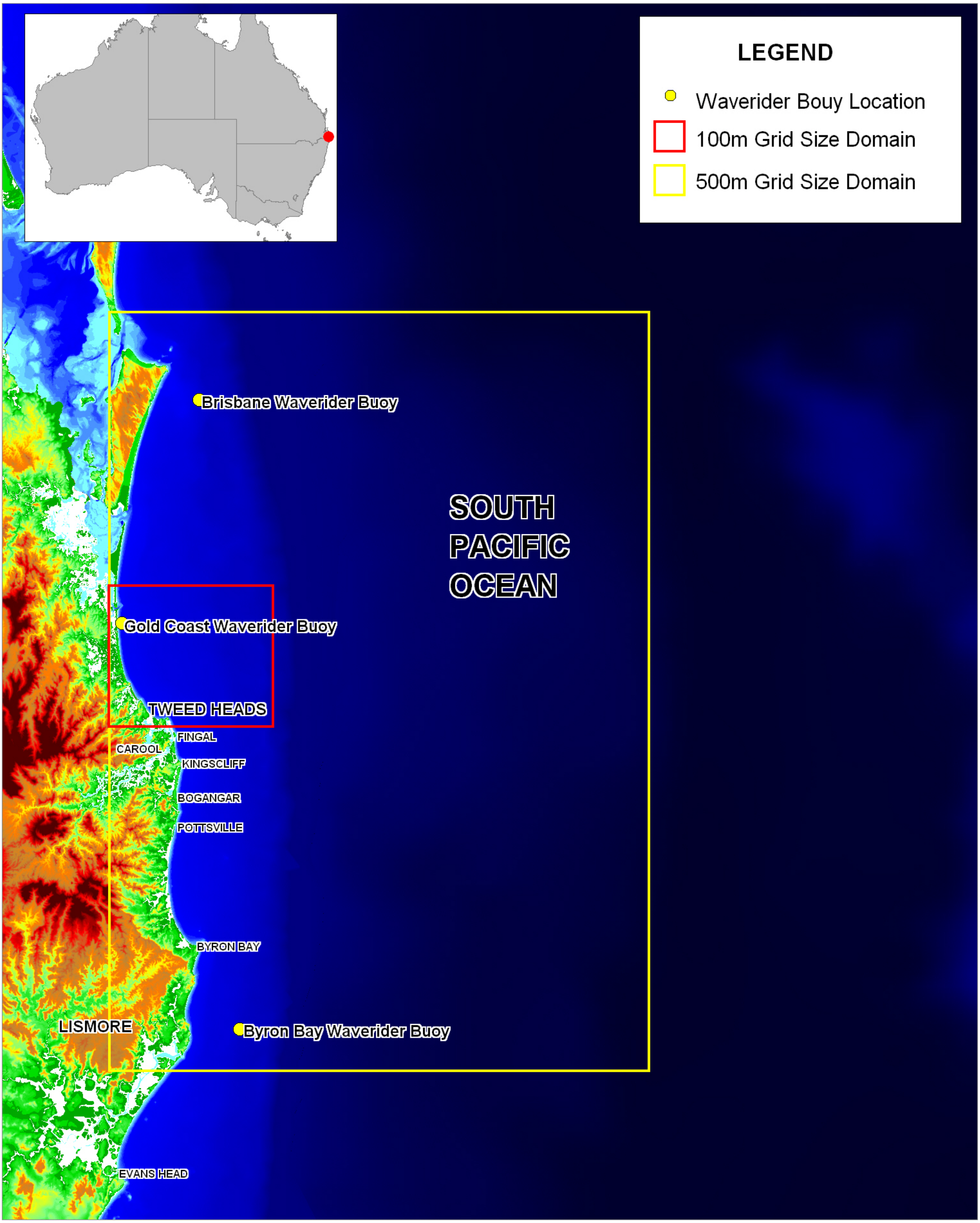
Due to this abundance in profile and wave data, the Gold Coast is being used as a test case location to verify the selected longshore and cross-shore models prior to their application at Wooli Wooli.

5.1.1 Wave Transformation

Wave transformation modelling using SWAN has been applied to transform deepwater wave inputs into the nearshore. The developed wave transformation model employed a 500m grid extending from North Stradbroke Island to Byron Bay. A nested domain utilizing a 100m grid size was then employed to represent the bathymetry from Coolangatta to South Stradbroke Island. The extent of the above mentioned domains are shown in Figure 5-1.

To validate the wave transformation model, modelling of wave rider buoy data from the Brisbane Buoy, offshore from North Stradbroke Island, was used to calculate wave height information at the Gold Coast Buoy, where concurrent recorded wave rider buoy data were available. Optimising of input parameters, such as directional spreading was done using a root mean squared method of comparison between the recorded and model wave heights from the Gold Coast for 2004 through till 2005. In addition to the 2004-2005 validation, further validation using the 3 month dataset for 2007 was used to further test the capabilities of the developed model to successfully refract waves to the Gold Coast buoy location.

Figure 5-2 and Figure 5-3 show the model validation results using wave data for 2004. The model results show that for the majority of cases the wave transformation successfully represents the Gold Coast wave data using Brisbane wave rider data to drive the model. Calculation of the mean wave height for the modelled period show good comparisons between the modelled and recorded data at the Gold Coast. The mean significant wave height for the modelled data and the recorded data at the Gold Coast was 1.09m and 1.05m respectively, whilst the Brisbane Waverider Buoy had a mean wave height of 1.55m for the modelled period.



Title:

Wave Transformation Model - Gold Coast

Figure: **5-1**

Rev: **A**

N

0 30 60km

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : I:\B15289_I_BRH Kyogle CLB\DRG\2008_Event\FLD_011_Rainfall_Distribution.WOR

Concurrent periods of data from the Brisbane Waverider Buoy, offshore in 73m depth of water, and the Gold Coast waverider buoy in 18m depth of water, were used for the wave transformation model calibration and validation

Analysis of the results show that for smaller wave heights (Hsig<0.5m) the wave transformation model does, for some periods, under predict wave heights. Exceedance plots of the model calibration results, shown in Figure 5-3, also show some variation in recorded wave heights compared with the and model results for significant wave heights around 4m in height. This is believed to be for the following reasons:

1 Small events of this magnitude are typically generated by local winds resulting in a large directional spreading. Within the wave transformation model a small shift in wave height may result in a large change in calculated wave height, especially from the northern and southern quadrants where wave refraction results in larger variation in wave height from the Brisbane and Gold Coast wave rider buoys. The wave transformation model cannot account for this.

2 Non-uniform generation of local wind waves may also result in recorded wave heights at the Gold Coast being greater than the modelled wave heights. The wave transformation model is only being driven by offshore wave forcings, no account is made for wind. If local winds are generating an increase in local wind waves between the Brisbane and Gold Coast Buoys, this will not be accounted for by the model.

Recognising these factors, Figure 5-2 and Figure 5-3 show good model calibration of the wave transformation model developed to represent the Gold Coast.

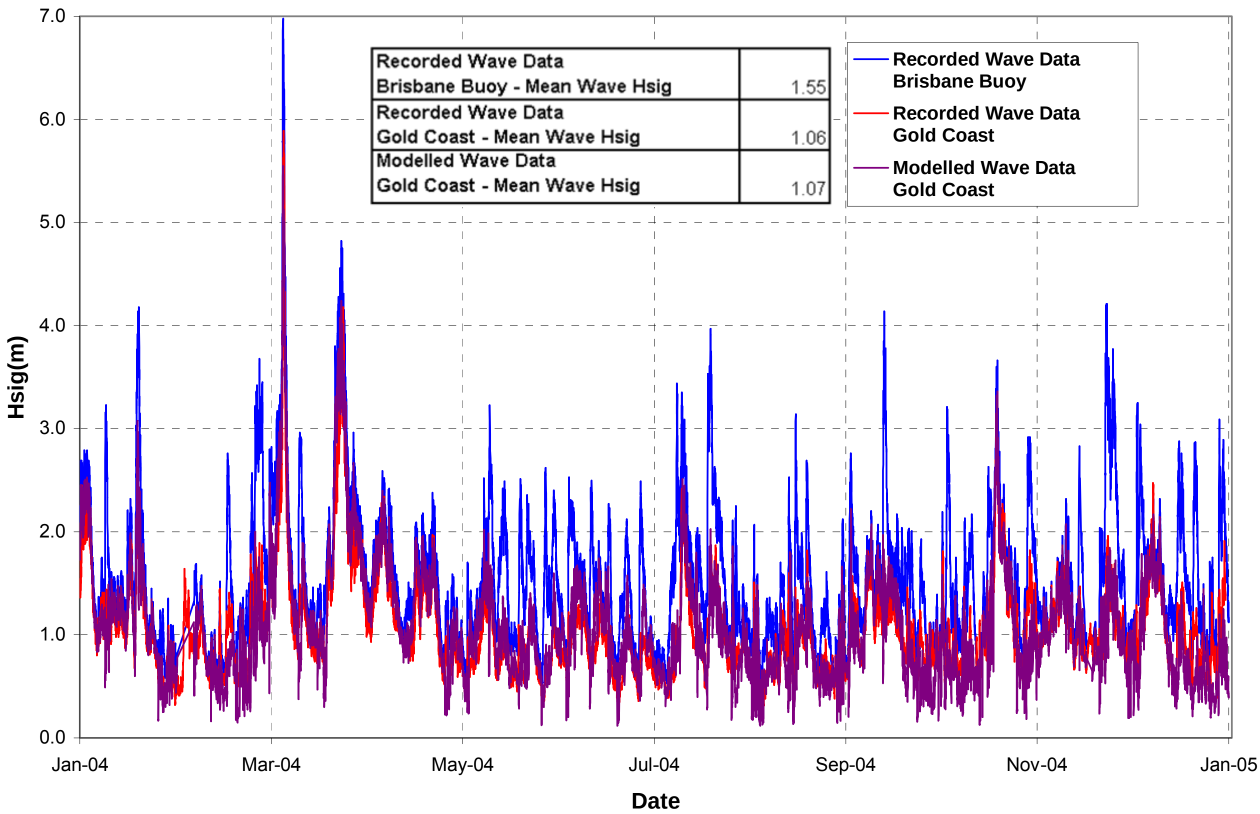


Figure 5-2 Wave Transformation Model Validation- Gold Coast 2004

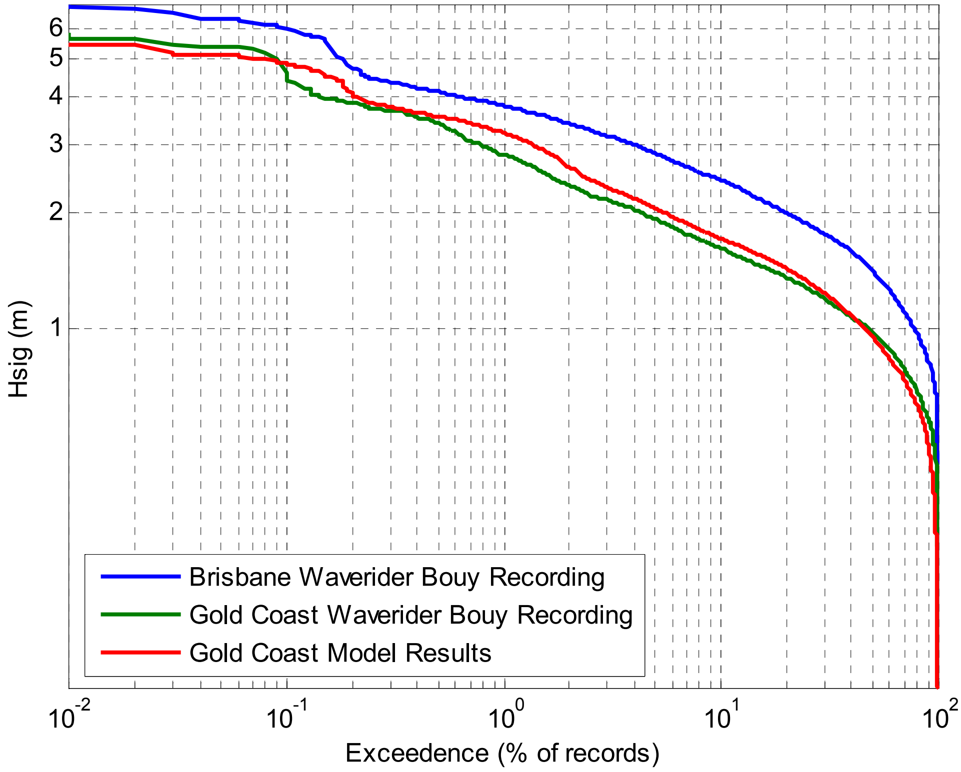


Figure 5-3 Wave Transformation Model Validation- Wave Height Exceedance Gold Coast 2004

Figure 5-4 shows the model results for the assessed 3 month period in 2007. The model results further support the model validation completed using the recorded 2004-2005 dataset.

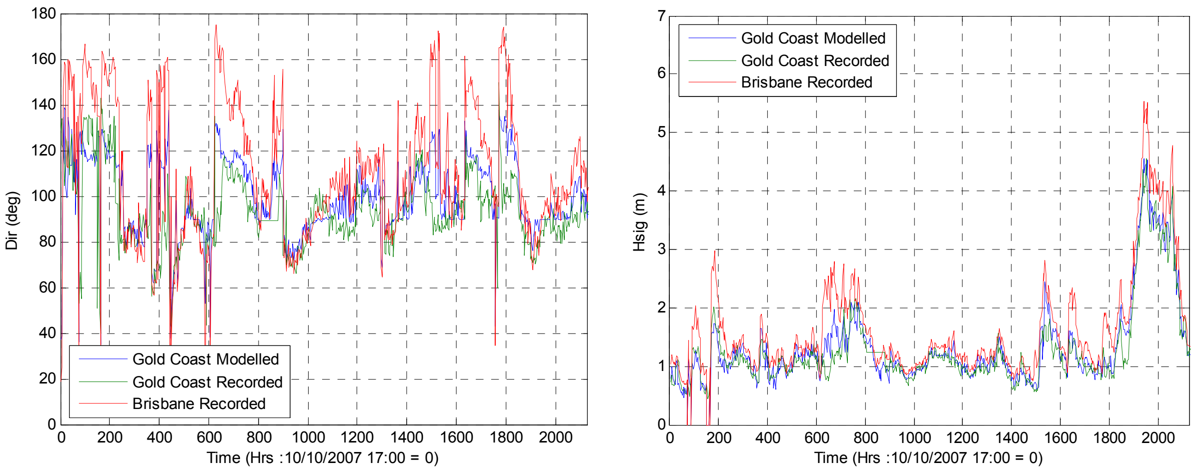


Figure 5-4 Wave Transformation Model Validation- Gold Coast 2007

5.1.2 Longshore Modelling

It has been historically calculated that the annual littoral drift at the Gold Coast is approximately 500,000m^3 (Patterson, 2007a, 2007b). Longshore modelling of the Gold Coast was undertaken for the period from 2000 to 2005. As a preliminary assessment the CERC formula (Equation 1) was used to estimate the LSMOD calibration parameter K. Although specific survey data were not available showing alongshore changes in shoreline orientation for this period the LSMOD modelling was undertaken with the aim to calibrate the model to produce a long-term average annual littoral drift of 500,000m^3.

Over the 2000 to 2005 period the shoreline from Burleigh Heads in the south to the Gold Coast Seaway in the north was in a stable orientation (i.e. No external influences were causing the reorientation of the beach).

LSMOD was tested for this period with the aim to check that the nearshore wave nesting procedure was adequately representing the actual wave conditions along the northern Gold Coast. Within the LSMOD model, if the model nesting of the wave transformation model does not accurately represent the variation in wave energy for the extent of the model, over long simulation periods (years), the model will show a reorientation of the shoreline to balance the incorrect input of either too much or too little wave energy for a given location within the model. If however, the nesting of the wave transformation model is representing the wave inputs correctly the shoreline modelled by LSMOD will maintain its current stable shoreline orientation.

The assessment of the model results for the 2000 to 2005 period illustrates that the nesting procedure is operating correctly, which has resulted in a stable shoreline orientation for the modelled scenario. Figure 5-5 shows the initial shoreline orientation. Relative to the initial shoreline position, Figure 5-6 shows the shoreline change results for the LSMOD modelling. In addition, Figure 5-7 shows the net sediment transport calculations and model results from the CERC equation assessment, compared to the LSMOD modelling. The calculated average values shown in Figure 5-7 are within the net littoral drift range calculated for the Gold Coast, documented within other previous coastal engineering studies (Patterson, 2007a, 2007b). Shown in Figure 5-7 is the variation in calculated values between the CERC equation and LSMOD is due to the CERC calculation using a fixed shoreline orientation, whilst the shoreline orientation within LSMOD varies dynamically with each simulated timestep.

The combination of the realistic variation in shoreline and the modelled littoral drift rate verify that the developed model is adequately representing the longshore transport processes influencing the shoreline response along the northern Gold Coast.

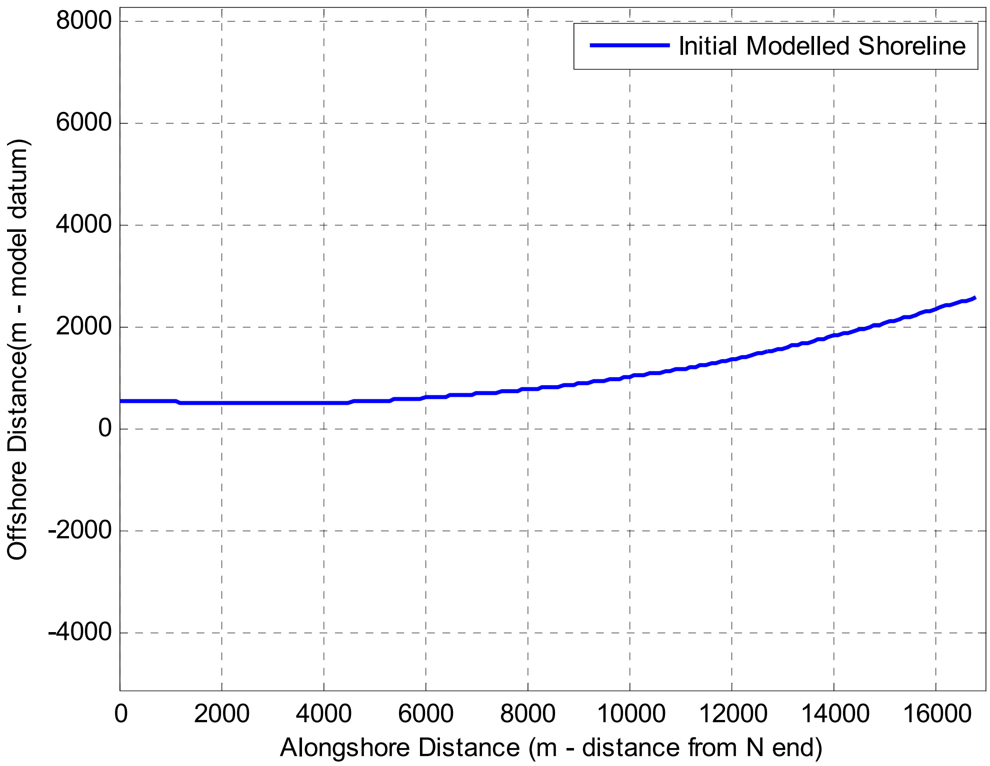


Figure 5-5 LSMOD Model Verification – Initial Shoreline Position

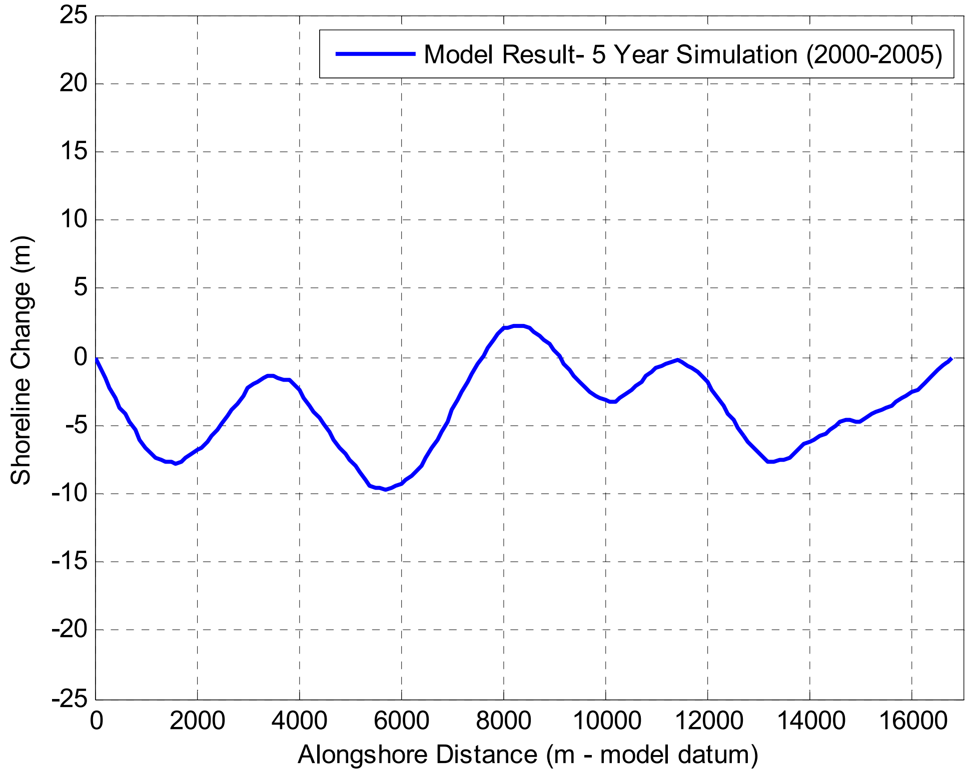


Figure 5-6 LSMOD Model Verification – Shoreline Change

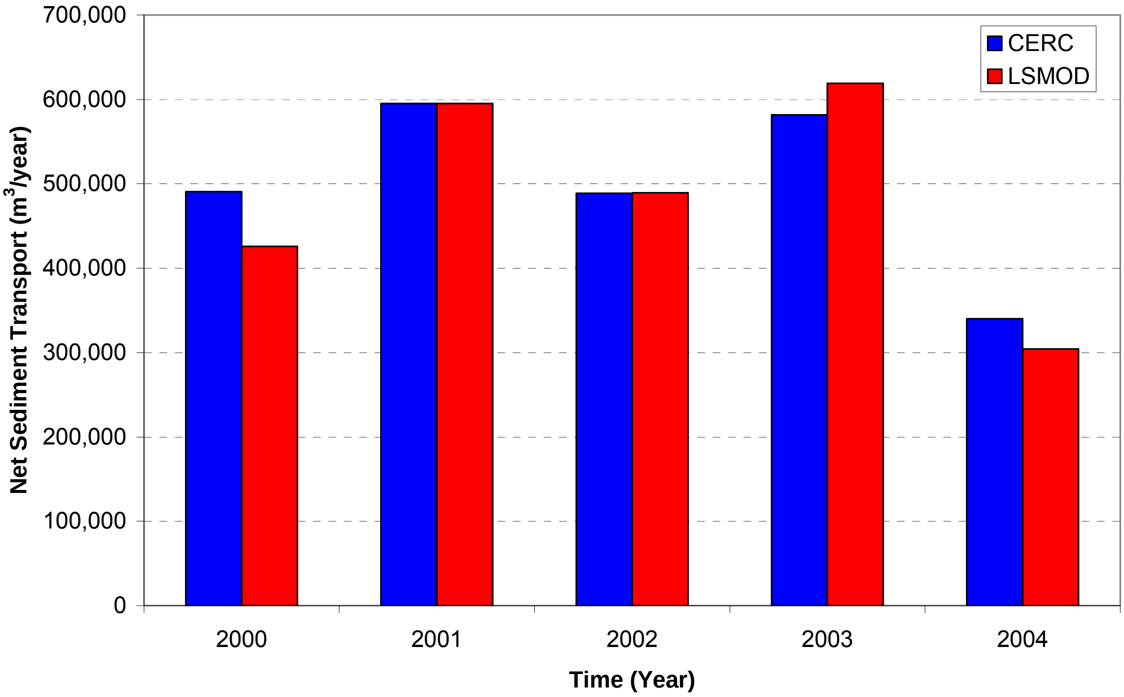


Figure 5-7 LSMOD Model Verification – Littoral Drift Comparison

5.1.3 Cross-Shore Modelling

The developed cross-shore model was validated for two significant erosion events, June to August 1967 and April to December 1988. These periods were selected for the following reasons:

1. The 1967 July erosion event represents the largest recorded erosion event experienced at the Gold Coast.
2. Surveyed profile information is available prior to and after the 1967 event.
3. The 1988 period experienced two moderate erosion events in May and November. Between these dates significant accretion occurred.
4. The 1988 period between April and December represents the most frequent recording period from the entire dataset available for the Gold Coast. Between these dates surveys were taken monthly.

5.1.3.1 Calibration Parameters

There are two types of calibration parameters required to run the developed geometric time stepping cross-shore model. These include profile parameters, which are required to ensure the correct representation of the cross-shore profile for the study area. Additionally, rate parameters governing the rate of change for accretion and erosion in the Crank Nicholson scheme are also used. Table 5-1 lists the calibration parameters and appropriate values used for the validation of XSMOD using the Gold Coast as a case study location.

Table 5-1 Cross-Shore Model Calibration Parameters – Gold Coast

Sediment/Profile Parameters		Rate Constant Parameters	
Grain Size (GS) = 0.22	[mm]	Erosion Rate Constant (ke) = $2e^{-6}$	[h^{-1}]
Deepwater Slope (m_2) = 60	[m/m]	Accretion Rate Constant (ka) = $8e^{-9}$	[h^{-1}]
Transition Slope (m_1) = 30	[m/m]		
Dune Slope (m_0) = 5	[m/m]		
Berm Height (B) = 5.5 ETA66, 4.58 ETA63 [m]			
Sediment Scale Parameter (A) = 0.105	[$m^{1/3}$]		
Bar Extent (h_b) =0.7	[-]		

5.1.3.2 Model Results

1967 Model Validation

As mentioned above, the 1967 July event is recognised as the largest erosion event experienced at the Gold Coast in living memory. The erosion event was the result of four cut off lows forming in quick succession offshore from the Gold Coast. The first low developed to the north of the Gold Coast on the 8th of July. The low of moderate strength tracked down the coast directly over the Gold Coast before dissipating on the 14th in the Coral Sea. The second low developed off Coffs Harbour on the 16th and tracked quickly north east towards New Caledonia. Four days later on the 20th of July a third low formed off Mackay. The low was of moderate strength and tracked south quickly, passing directly over the Gold Coast. The final cut off low formed directly offshore from the Gold Coast on the 24th. This low was extremely large and intense. Tracking south-east the low developed an extremely tight pressure gradient with a neighbouring high pressure system located over Victoria/southern NSW. This resulted in huge seas off the Gold Coast. In addition to the high frequency of storm events over this July period, the timing of the events, coinciding with moderately high tides made the month long erosion event extremely damaging to the coastline along the Gold Coast.

Due to the lack of wave data, the 1967 erosion event was modelled using hindcast wave inputs derived from synoptic charts obtained from the QLD Bureau of Meteorology. Accounting for the possible inaccuracies in the input wave data, based on hindcast calculations, the model produced very good results.

Figure 5-8 shows the modelled shoreline position from June to August 1967.

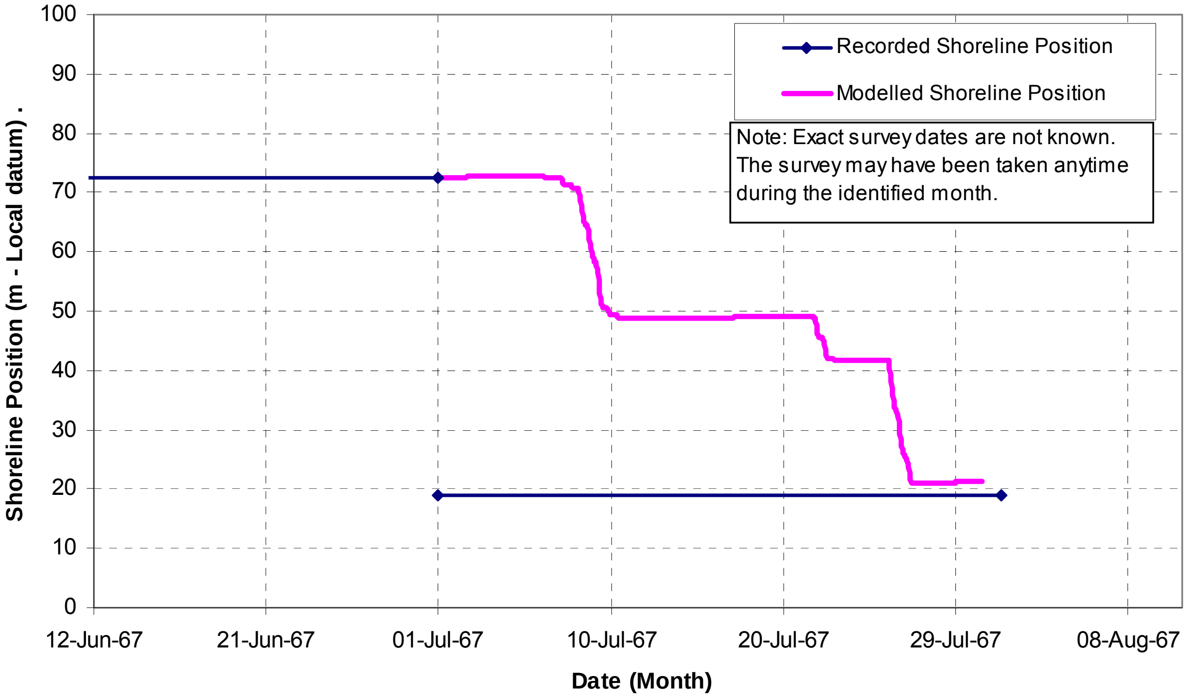


Figure 5-8 1967 Cross-Shore Model Validation – Shoreline Position ETA66

1988 Model Validation

The developed cross-shore model was also verified using data representing the period from April 1988 to December 1988. During this period two moderately sized erosion events occurred in May and November 1988. Between these two erosion events milder swell conditions were experienced, resulting in shoreline accretion. Assessment of the model results show good correlation to the recorded profile data. The model results are shown in Figure 5-9.

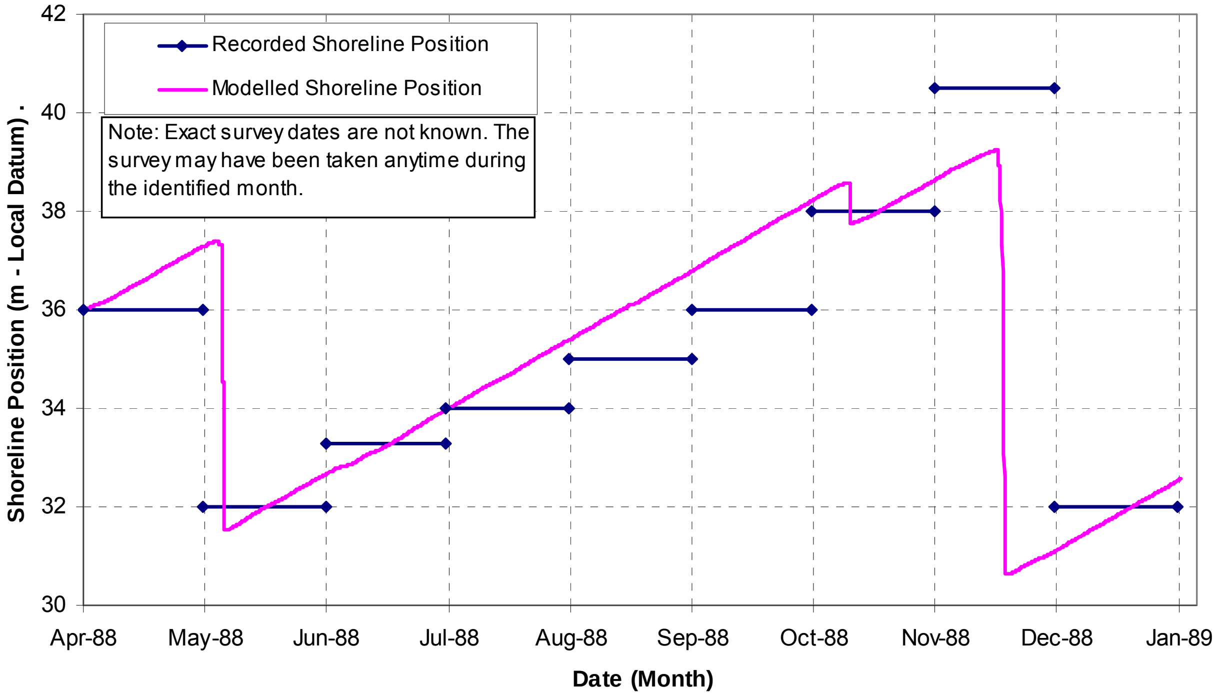


Figure 5-9 1988 Cross-Shore Model Validation – Shoreline Position ETA63

5.2 Model Comparisons

In addition model validation using the 1967 and 1988 datasets from the Gold Coast, a further assessment comparing modelled shoreline recession resulting from an instantaneous increase in water level with no change in wave forcings was conducted in parallel to a Bruun Rule assessment.

Model comparisons with Kriebel and Dean (1985), Kriebel and Dean (1993) and Miller and Dean (2004) have also been completed.

5.2.1 Bruun Rule Shoreline Recession

Testing of the XSMOD versus the Bruun Rule has been undertaken to verify that the XSMOD model can predict shoreline recession due to water level forcing correctly.

Based on available storm profile data for the Gold Coast a Bruun rule assessment was undertaken to predict possible shoreline recession resulting from a one metre increase in water level. Figure 5-10 shows a portion of the available data for the Gold Coast. Based on the profile data, using a depth of closure of 15m depth, a berm height of 5.5m elevation and an active surf zone width of 891.5m, the Bruun Rule predicted a shoreline recession of 43.5m for an increase in water level of 1m.

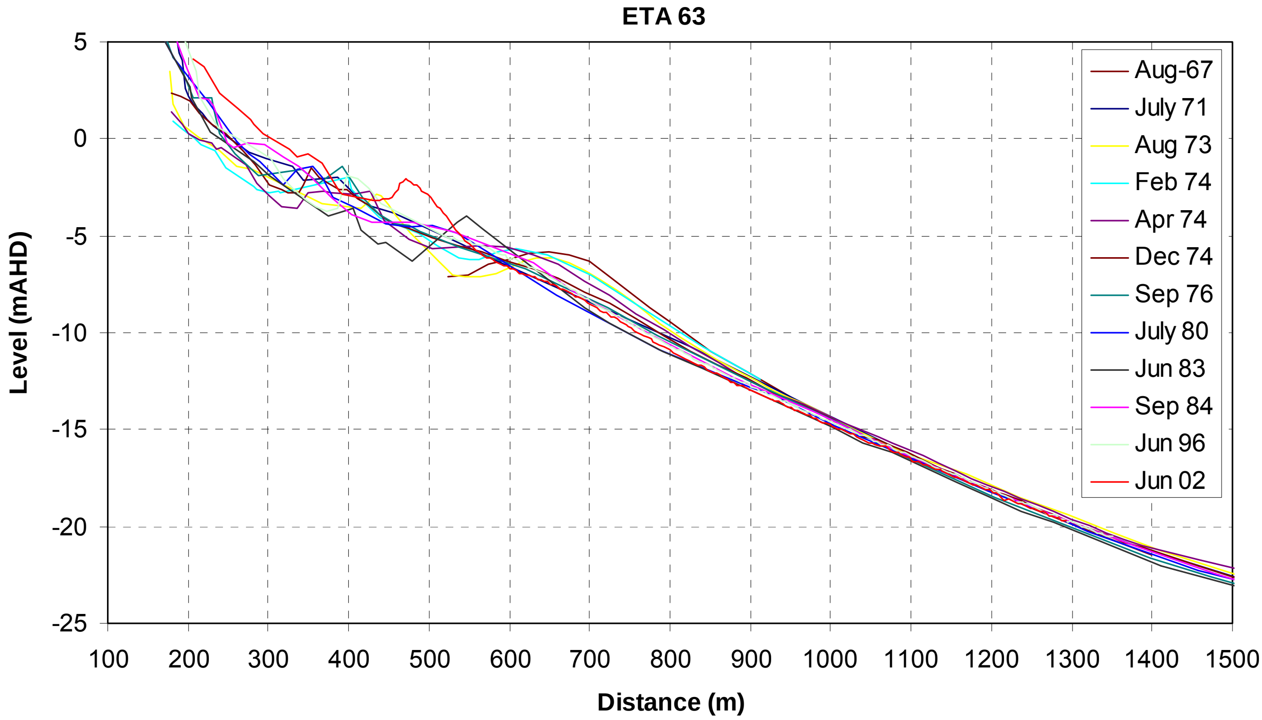


Figure 5-10 Gold Coast Profile Data

Of the available profile data the August 1967 profile survey exhibited both the greatest storm bite and the most offshore bar location. Using the 1967 profile as the initial profile, the developed cross-shore model was run for an indefinite period using constant wave forcings and an increase in water level of one metre. The test case using the developed cross-shore model calculated a shoreline recession of 40.4m resulting from a linear incremental increase in water level over 100 years by 1m (10mm per year). These results are comparable to the predicted shoreline recession calculated using the Bruun Rule. Figure 5-11 and Figure 5-12 show the model results for the basic shoreline recession test.

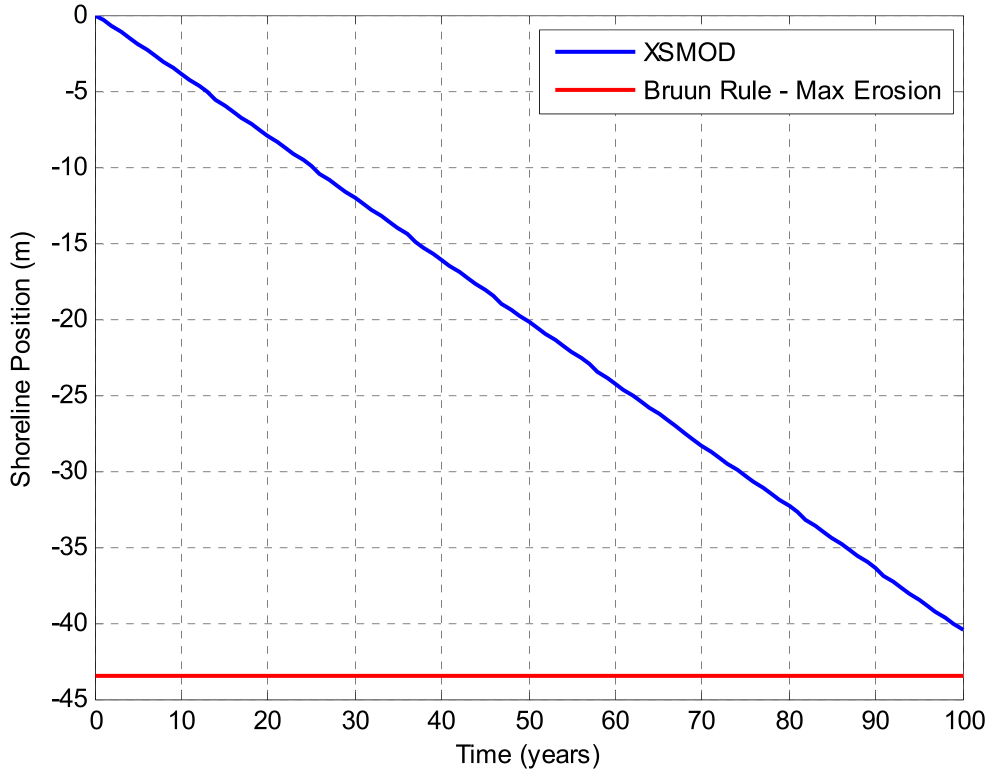


Figure 5-11 XSMOD Shoreline Recession Test – Shoreline Position

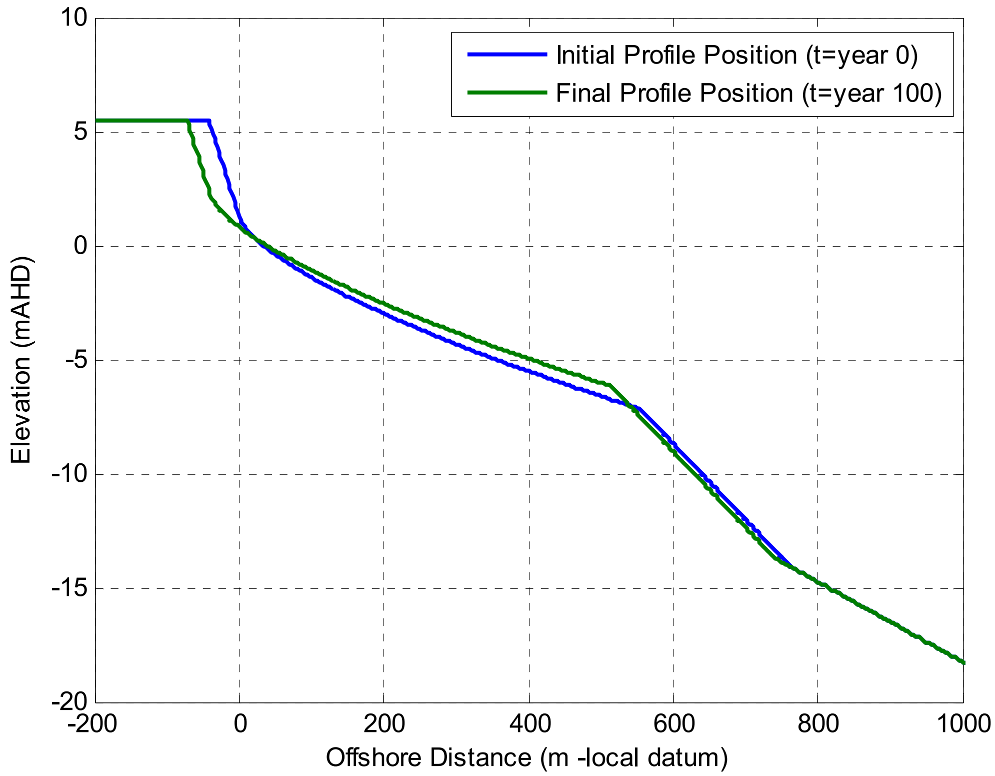


Figure 5-12 XSMOD Shoreline Recession Test – Profile Position

5.2.2 Kriebel and Dean 1993

Prior to the development of XSMOD, the Kriebel and Dean convolution model (Kriebel and Dean, 1993) was tested against the recorded survey data from the Gold Coast for 1976 and 1988. The Kriebel and Dean convolution model was developed as a shoreline erosion model, suitable for application to isolated erosion events.

Initial testing of the Kriebel and Dean convolution model used the methodology applied by Callaghan *et al.* (2008). This methodology extended the application of the original Kriebel and Dean convolution model to accommodate for multiple erosion events. During the Kriebel and Dean assessment, the following accretion parameters have been used;

- Accretion rate constant = 400 hours (0.0025hrs^{-1})
- ETA66 Accretion $R_{\infty,a}$ = 52m (Local Datum).
- ETA63 Accretion $R_{\infty,a}$ = 72.6m (Local Datum).

The accretion rate constant value matches the values used by Callaghan *et al.* (2008) for the Narrabeen Beach assessment. The Accretion $R_{\infty,a}$ value has been estimated based on accretionary ambient wave conditions.

The Kriebel and Dean convolution model was tested for the periods described in Section 5.1.3.2, July 1967 and April 1988 to December 1988. The model results using the above mentioned accretion parameters are shown in Figure 5-13 and Figure 5-14. Figure 5-15 and Figure 5-16 compare the results of the convolution model verses XSMOD and recorded survey data.

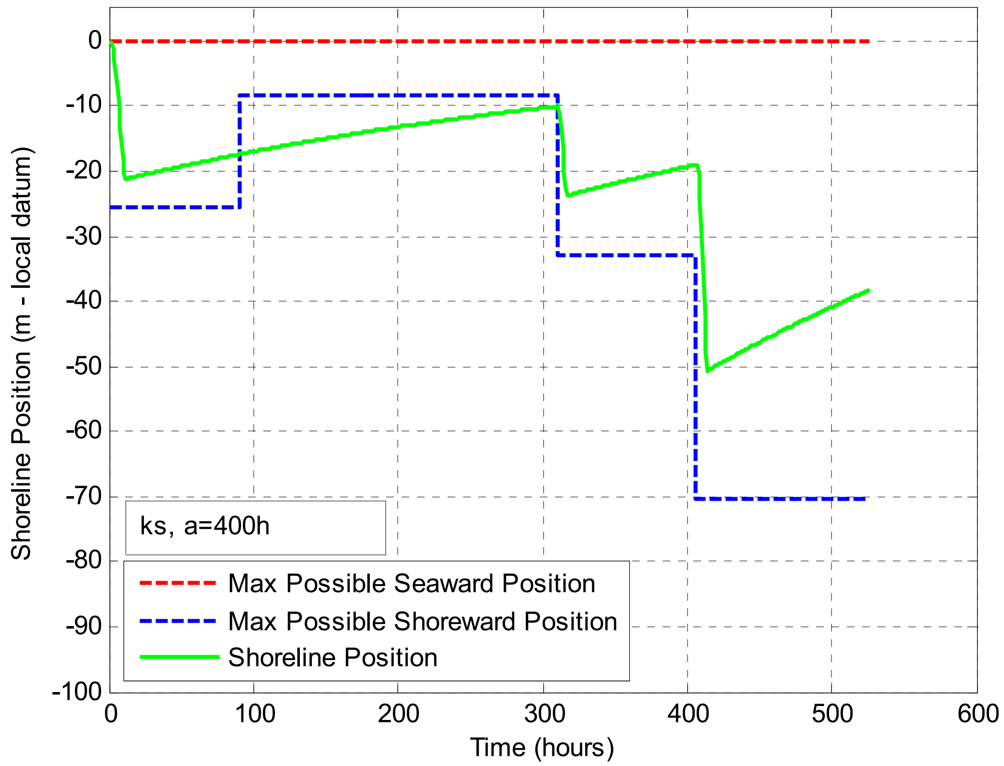


Figure 5-13 Kriebel and Dean (1993) Model Results - Gold Coast 1967 ETA66

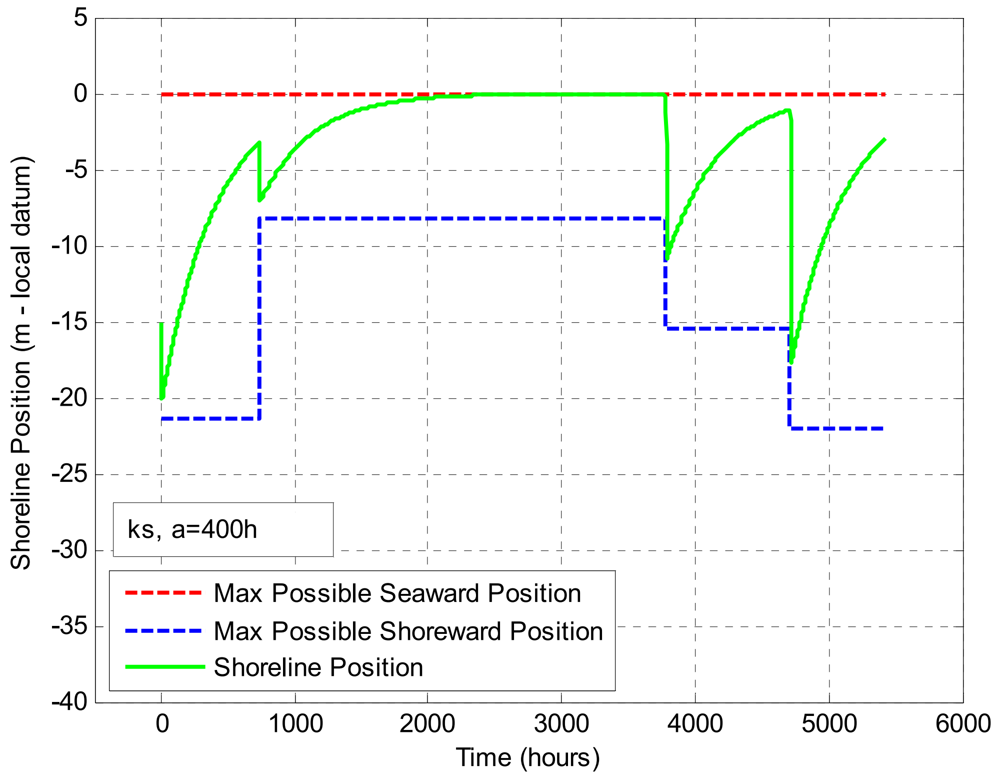


Figure 5-14 Kriebel and Dean (1993) Model Results - Gold Coast 1988 ETA63

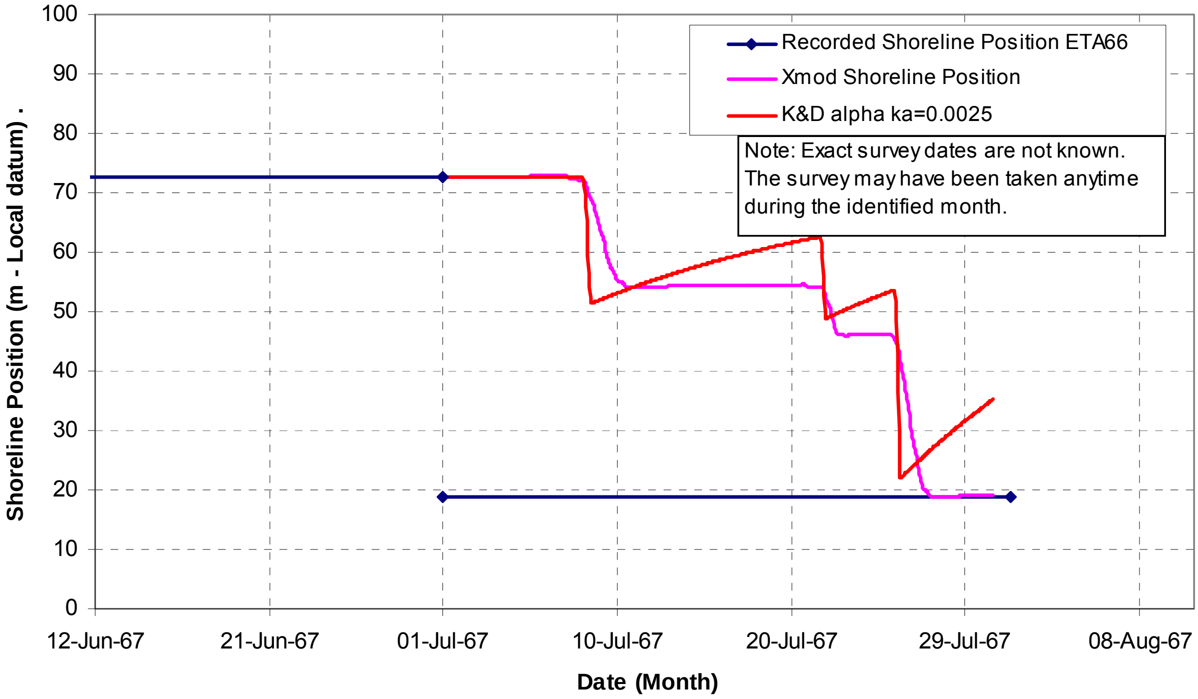


Figure 5-15 Kriebel and Dean (1993) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1967

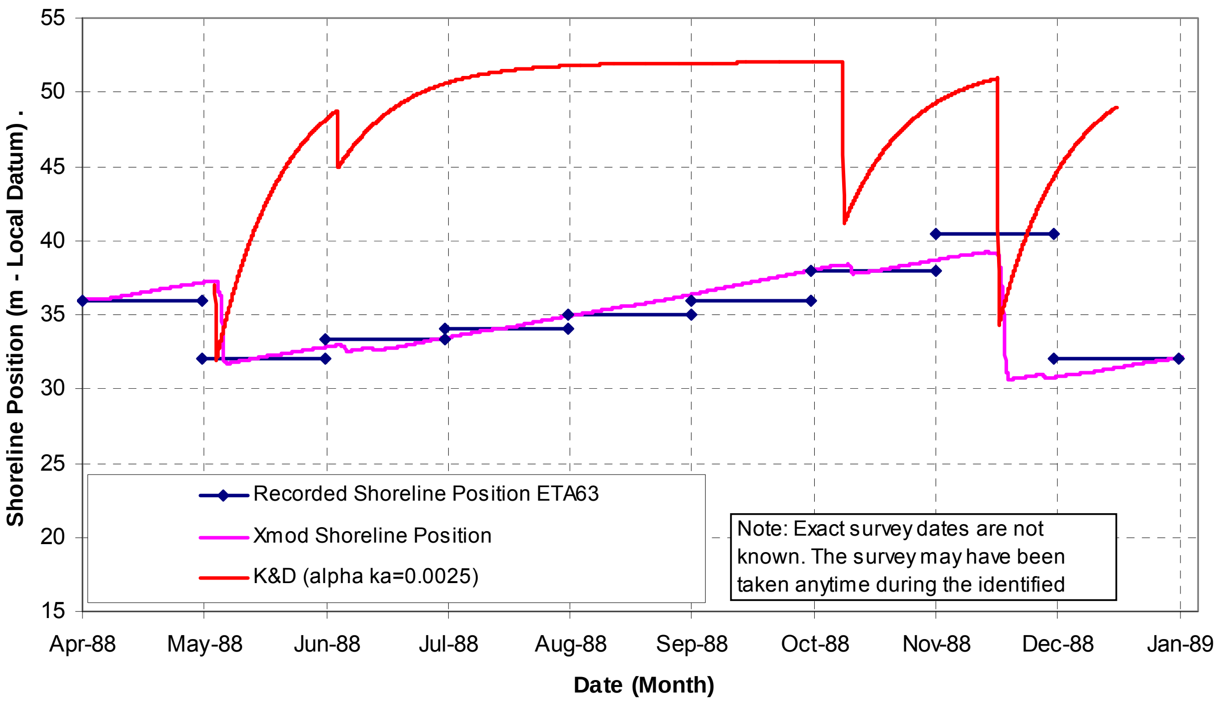


Figure 5-16 Kriebel and Dean (1993) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1988

Figure 5-15 shows the convolution model to represent the shoreline erosion resulting from the multiple events in July 1967 to be closely predicted. However, based on historic survey data from the Gold Coast, the rate of accretion appears to respond much too quickly.

Figure 5-16 shows the convolution model results for 1988. The model results do not represent the recorded shoreline position at any point.

Further testing of the convolution model was undertaken with some modifications to the accretion parameter assumptions. The following assumptions were made;

- Accretion rate constant = 9000 hours (0.0001hrs^{-1}) equal to the calculated accretion rate used by the XSMOD model.
- ETA66 Accretion $R_{\infty,a}$ = 52m (Local Datum) – unchanged from the previous assessment.
- ETA63 Accretion $R_{\infty,a}$ = 72.6m (Local Datum) – unchanged from the previous assessment.

The model results using the above mentioned accretion parameters are shown in Figure 5-17 and Figure 5-18. Figure 5-19 and Figure 5-20 compare the results of the convolution model versus XSMOD and recorded survey data.

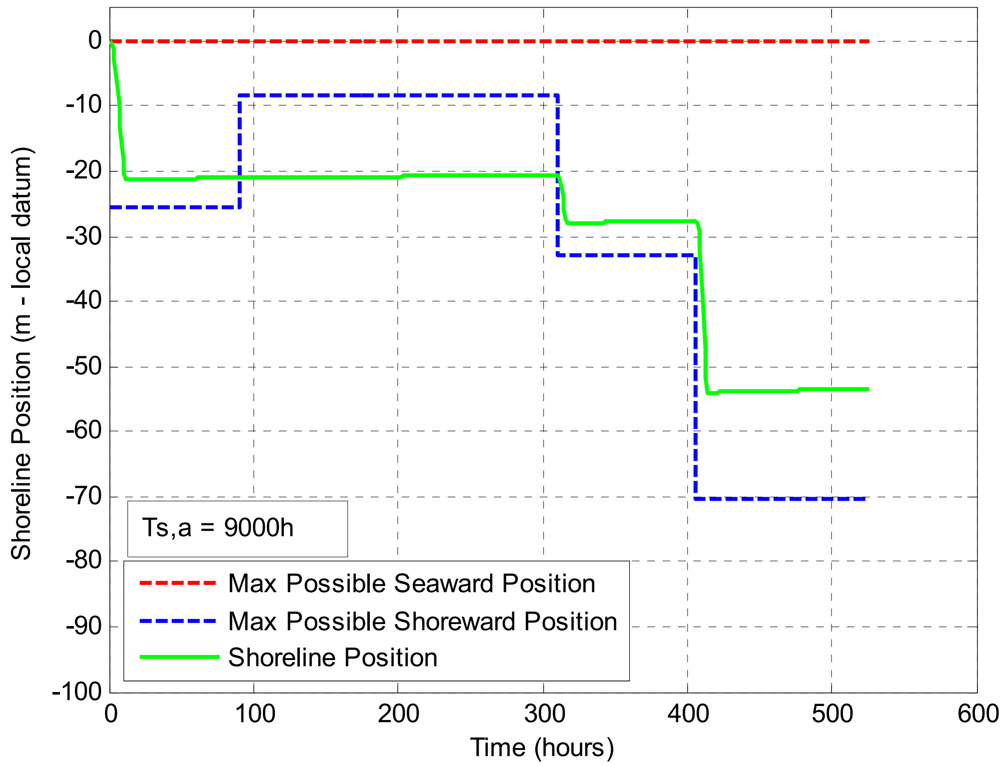


Figure 5-17 Kriebel and Dean (1993) Model Results - Gold Coast 1967 ETA66

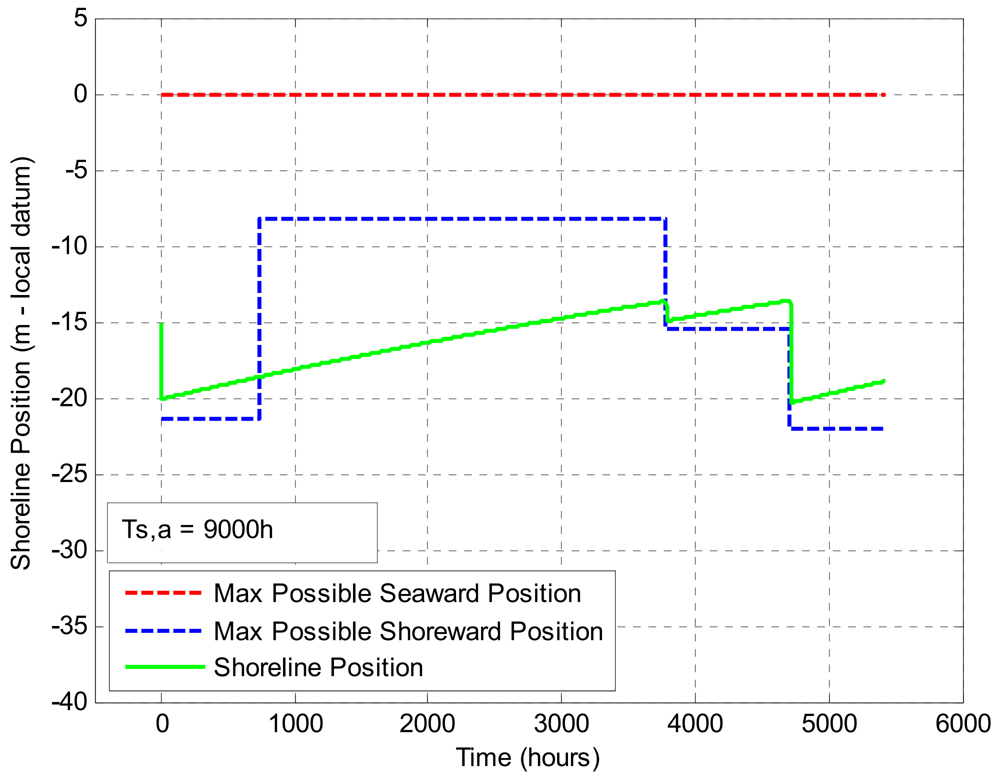


Figure 5-18 Kriebel and Dean (1993) Model Results - Gold Coast 1988 ETA63

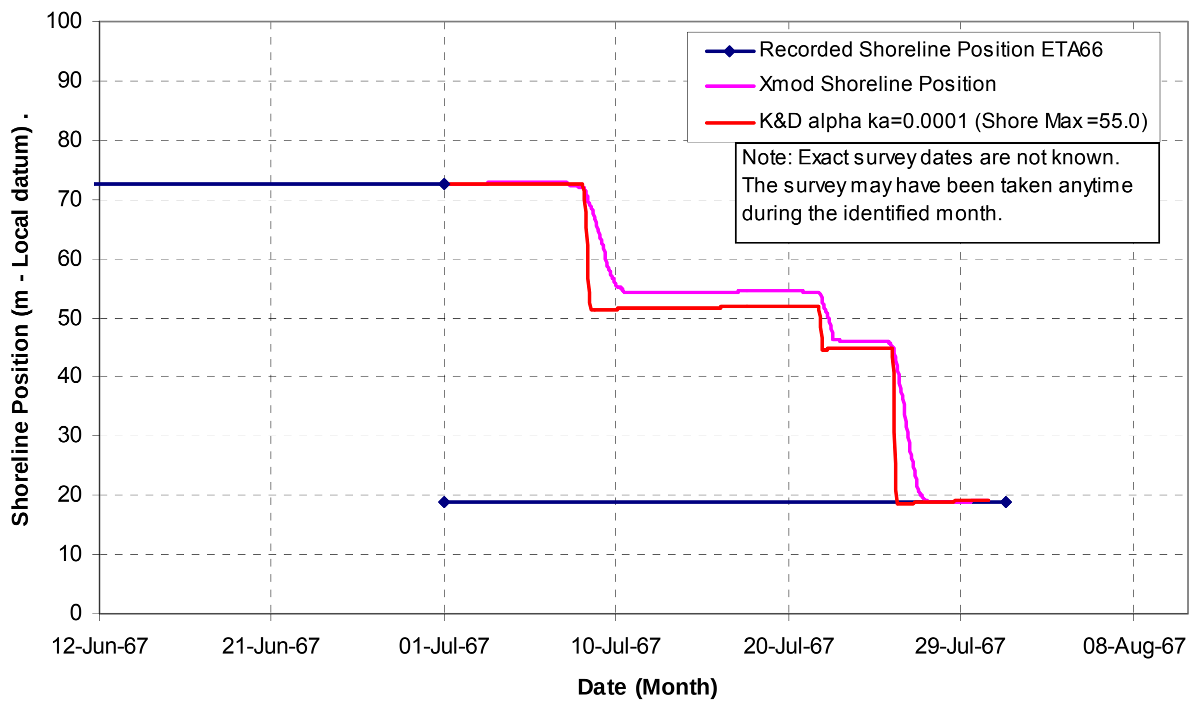


Figure 5-19 Kriebel and Dean (1993) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1967

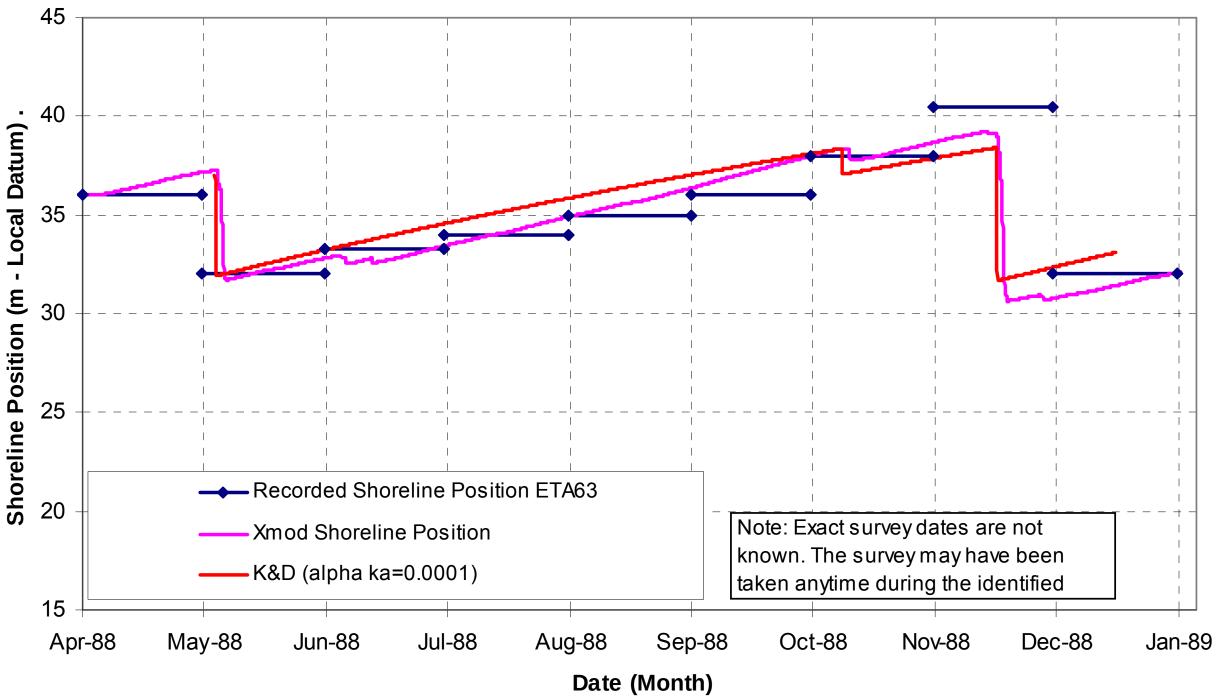


Figure 5-20 Kriebel and Dean (1993) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1988

Figure 5-19 and Figure 5-20 show the results from the convolution model assessment. The 1967 and 1988 results show good agreement with the recorded survey data.

The model results for the convolution model assessment are similar to the XSMOD model results. However, it is believed that the following methodologies required when applying the Kriebel and Dean convolution method to multiple events may result in the accumulation of error over time:

- Definition of a threshold value to define individual storm events from accreationary periods. and
- The fixed maximum accreted shoreline position.

To explain further, if a storm threshold value of 3.0m Hsig is chosen, for a fully accreted beach state a storm event resulting in wave heights of 2.99m will not be considered a storm event capable of producing erosion. If the storm event were to produce waves of 3.01m, the multiple event convolution modelling approach will predict erosion. The selection of the threshold value is likely to have a significant impact of predicted erosion volumes.

Additionally, the application of a fixed value for the maximum accreted shoreline position is unrealistic. Periods of accretion occurring during moderate wave activity compared with alternate periods during milder wave activity will result in different accretion characteristics. In reality the value representative of the maximum accreted shoreline position at any given time is dynamic, dependent on the wave conditions for the given accretion period.

The XSMOD model does not rely on the above mentioned limitations. During long term simulation this represents a significant improvement to the Kriebel and Dean convolution model.

XSMOD does not require a threshold value to define whether erosion or accretion should occur for a given timestep. XSMOD identifies erosion or accretion periods dependent upon the resultant profile represented for the previous timestep relative to the current timestep wave height and water level forcings. This approach is prefered to the user defined storm threshold value.

In terms of accretion modelling, using the time stepping approach, XSMOD can account for seasonal variations in ambient wave climate. This results in the dynamic prediction of the maximum accreted shoreline position for each timestep throughout the simulation period. Similarly, this approach is prefered to the user defined fixed maximum accreted shoreline position.

These features make XSMOD a powerful model, able to predict shoreline change over long timescales in response to input parameters of all time scales (half hourly to decades).

5.2.3 Miller and Dean

The Miller and Dean model (2004) was tested against the recorded survey data from the Gold Coast for 1976 and 1988.

Accretion and erosion rate parameters calibrated for the XSMOD model, listed in Table 5-1, were applied during the Miller and Dean assessment. The model result for the Miller and Dean model are shown in Figure 5-21 and Figure 5-22.

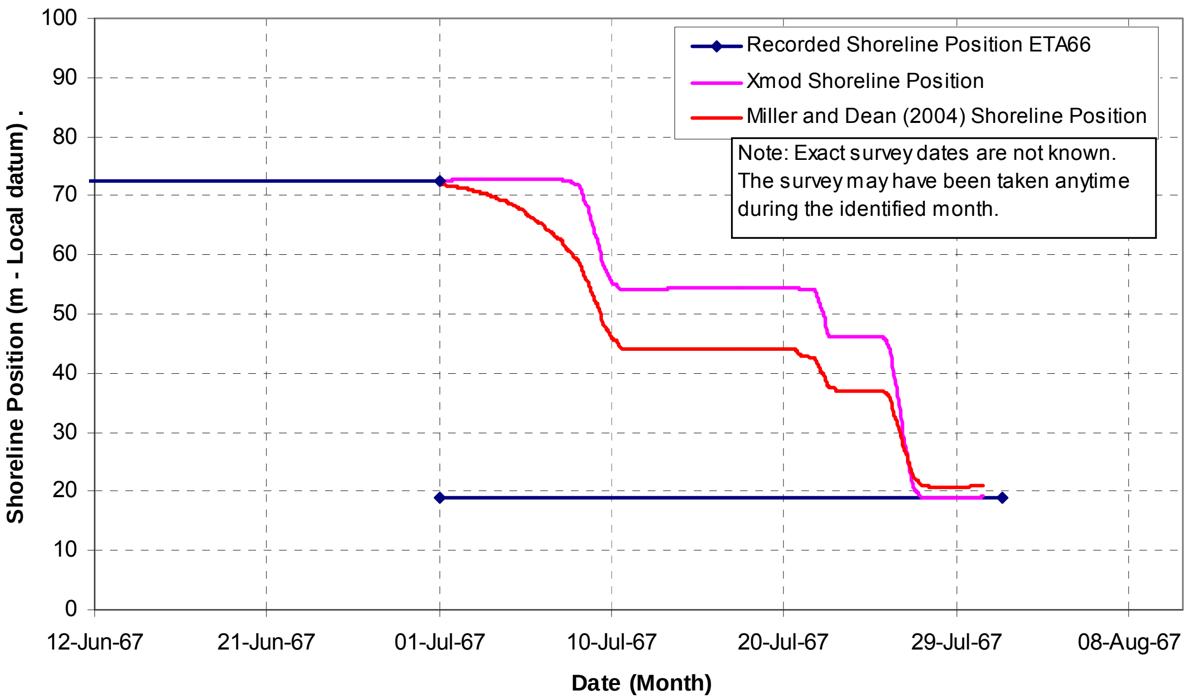


Figure 5-21 Miller and Dean (2004) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1967

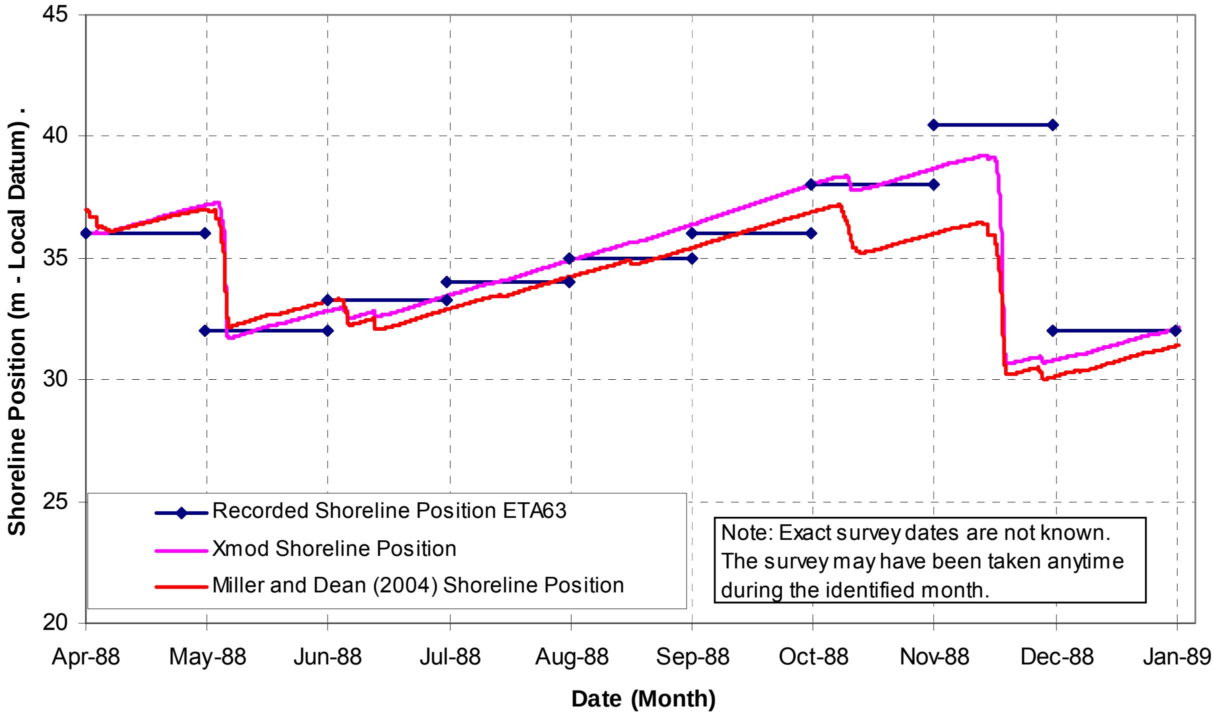


Figure 5-22 Miller and Dean (2004) vs XSMOD Model Comparison – Shoreline Position Gold Coast 1988

Overall the model results for the testing of the Miller and Dean approach are similar to the results of the XSMOD model. The Miller and Dean predictions are in general agreement with the recorded survey data. The variations in the modelled shoreline position when comparing the Miller and Dean approach verses XSMOD are directly related to the differences in predicted maximum equilibrium shoreline position calculated via the developed geometric approach and calculated using Equation 16.

In addition to the above testing, a sea level rise assessment identical to the Bruun rule assessment of the XSMOD model (1m increase in water level over 100 years, B=5m) was undertaken for the Miller and Dean model (2004). As has been discussed in Section 3.3.3.4, the Miller and Dean model (2004) is an evolving shoreline model which only accounts for sediment movement shoreward of the breakpoint. Due to the model not accounting for conservation of mass to the depth of closure (beyond the breakpoint) it was expected that the Miller and Dean model may underestimate shoreline recession during climate change assessments accounting for sea level rise.

Model results for the sea level rise assessment using the Miller and Dean model (2004) predicted a shoreline recession of 29.6m after 100 years. As was expected, this is an under prediction compared with the generalised Bruun rule and XSMOD sea level rise results.

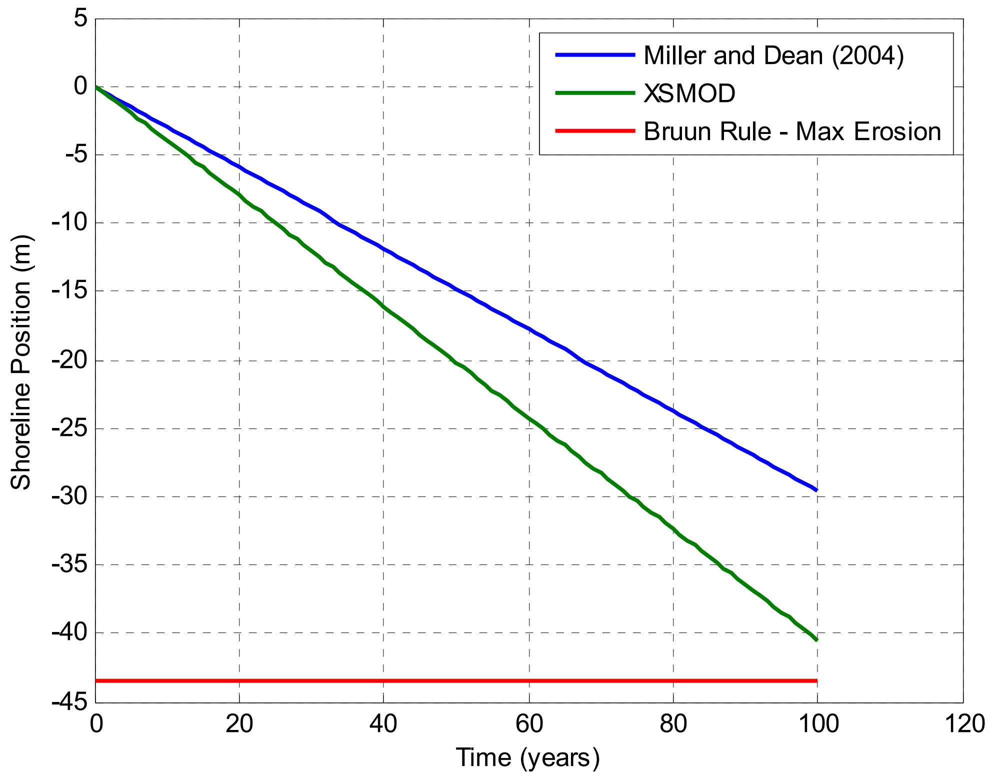


Figure 5-23 XSMOD Shoreline Recession Test – Shoreline Position

Overall, the Miller and Dean approach produced satisfactory results for the model validation periods of 1967 and 1988. The results show that the Miller and Dean approach may be suitable for multiple storm event simulations. It is believed the model results may have produced a closer match to the recorded data had different erosion and accretion rate parameters been used (i.e. Not based on the k_e and k_a identified for XSMOD).

The sea level rise assessment of the Miller and Dean model indicate the profile equation utilised by the approach may underestimate shoreline recession resulting from long-term sea level rise. These results highlight the inadequacies of the Miller and Dean model (2004) when applied to long term climate change scenarios assessing the impact of increases in sea level.

Based on the above assessment, the XSMOD model, developed as part of this Masters study, has been identified as the most appropriate modelling approach applicable for shoreline change studies assessing the impacts of multiple storm events and long-term changes in water level within the one modelling system.

6 MODEL APPLICATION

6.1 Wooli Wooli

6.1.1 Background

Located on the north coast of NSW, Wooli Wooli beach stretches for approximately 7km from Wilson Headland in the North to the Wooli Wooli River and Tree Point in the south. Along the north of Wooli Wooli Beach significant frontal dunes up to 15mAHD in height dominate the shoreline. Heading south the dunes decrease in size to around 5mAHD at the Wooli Wooli River. Figure 6-1 to Figure 6-3 show some pictures of Wooli Wooli Beach taken in early 2008. To the south of Wooli Wooli Beach the nearshore reef, located approximately 2km offshore from the river mouth, and North Solitary Island, located an additional 10km offshore, result in complex wave sheltering and focusing patterns along Wooli Wooli Beach.

Figure 6-1 North Wooli Wooli Beach 2008

Figure 6-2 Central Wooli Wooli Beach 2008

Figure 6-3 South Wooli Wooli Beach 2008

Historically, since the release of land along the Wooli Wooli peninsula in 1924, Wooli Wooli Beach has experienced three significant erosion events. These erosion events occurred in 1954, 1974 and

1996. Along the beach section adjacent to the Wooli water tower the erosion scarp from the 1996 event is still evident, as is shown in Figure 6-2. Some properties in Wooli are located on the frontal dunes, as little as 10 to 15m back from the 1996 erosion scarp. With minimal dune protection for these properties, the town of Wooli is recognised as one of the most at risk locations in NSW for coastal hazard.

Irregular photogrametry surveys of Wooli Wooli Beach have been undertaken for the period since 1942. In most cases these surveys extended only as far seaward as the approximate high-water mark. Cross section plots of the north, central and southern parts of Wooli Wooli Beach are shown in Figure 6-4 to Figure 6-6.

Analysis of the figures shows that between 1942 and 1996 the shoreline (at 4mAHD) has shifted landward approximately 20 to 30m for the full length of Wooli Wooli Beach. However, careful inspection of the survey data shows that the erosion events resulting in the greatest setback for north Wooli Wooli beach do not coincide with those for the southern end of Wooli Wooli beach. Figure 6-4 shows little variation in shoreline position for North Wooli Wooli Beach between 1942 to 1966, and 1978 to 1996. This implies that the North Wooli Wooli Beach experienced its greatest setback during the 1974 erosion event. In comparison, Figure 6-5 shows that the central section of Wooli Wooli Beach experienced the most significant erosion during the 1954 and 1996 erosion events. Similarly, Figure 6-6 shows the most significant erosion event for the southern section of Wooli Wooli Beach to be in 1996. The setback values using 1942 as the zero datum are listed in Table 6-1 for the northern , central and southern section of Wooli Wooli beach.

This variation in alongshore shoreline response to storm forcings represents an important component of this climate change study. To accommodate this, modelling procedures capable of representing this aspect of shoreline change have been developed.

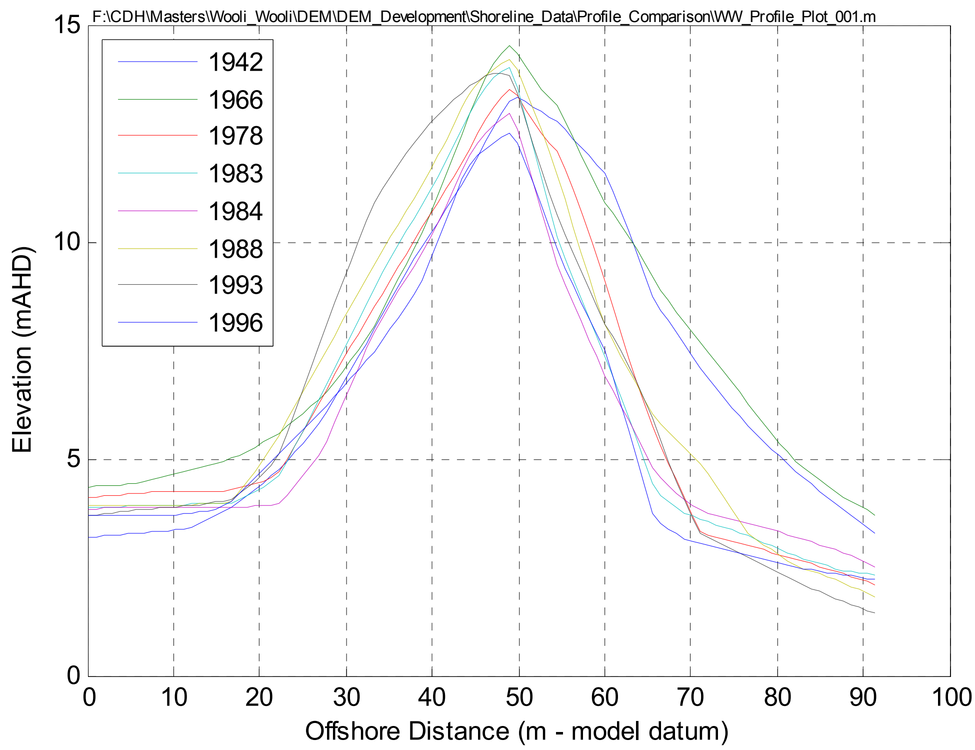


Figure 6-4 North Wooli Wooli Beach Profile Comparison

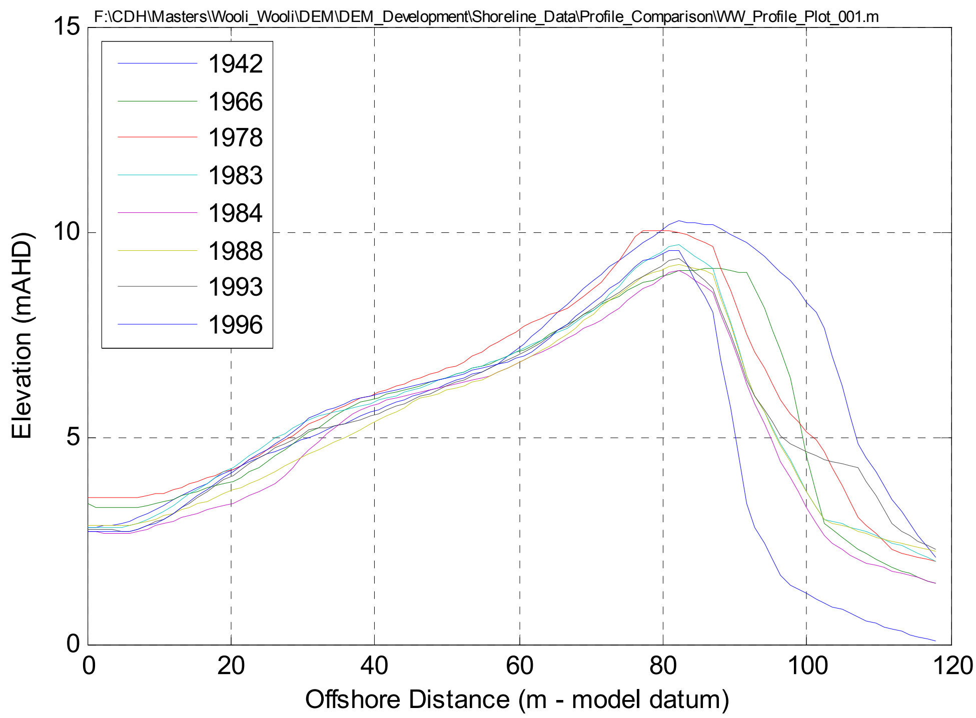


Figure 6-5 Wooli Wooli Beach Profile Comparison

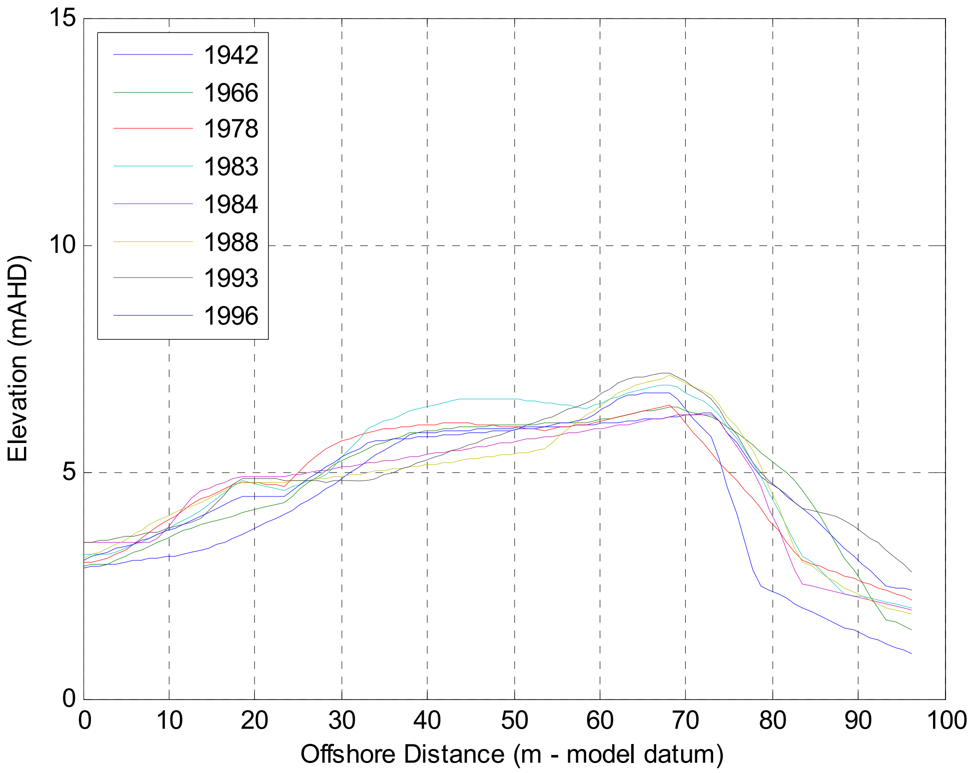


Figure 6-6 South Wooli Wooli Beach Profile Comparison

Table 6-1 Historic Shoreline Response – Wooli Wooli Beach

Year	North	Central	South
1942	0.0	0.0	0.0
1966	+2.8	-10.6	-1.9
1978	-19.4	-6.0	-4.9
1983	-18.5	-3.6	-6.8
1984	-14.8	-13.1	-8.7
1988	-13.8	-2.4	-6.8
1993	-19.4	+1.0	+3.9
1996	-23.1	-22.6	-13.6

6.1.2 Wave Data

Before modelling of the shoreline can be undertaken, long-term recorded wave and water level data is required to drive the sediment transport models. The Manly Hydraulic Laboratory has provided Waverider buoy data for Byron Bay. The provided data consisted of non-directional data for the period between 14/10/1976 to 26/10/1999 and directional wave data from 26/10/1999 to 31/03/2008. It was initially thought that the directional wave data provided would be able to be used directly to drive the sediment transport models for Wooli Wooli (with application of a wave transformation model). However, assessment of the directional data identified large gaps in the Byron Bay Buoy wave data. Comparisons with the Brisbane and Coffs Harbour wave rider buoy data identified that the gaps in the dataset consistently occurred during larger swell events. Figure 6-7 and Figure 6-8 show the directional Byron Bay data with the data gaps highlighted. Table 6-2 shows the corresponding recorded mean and maximum significant wave heights recorded by the Brisbane Buoy for the periods when the Byron Bay Buoy was non-operational.

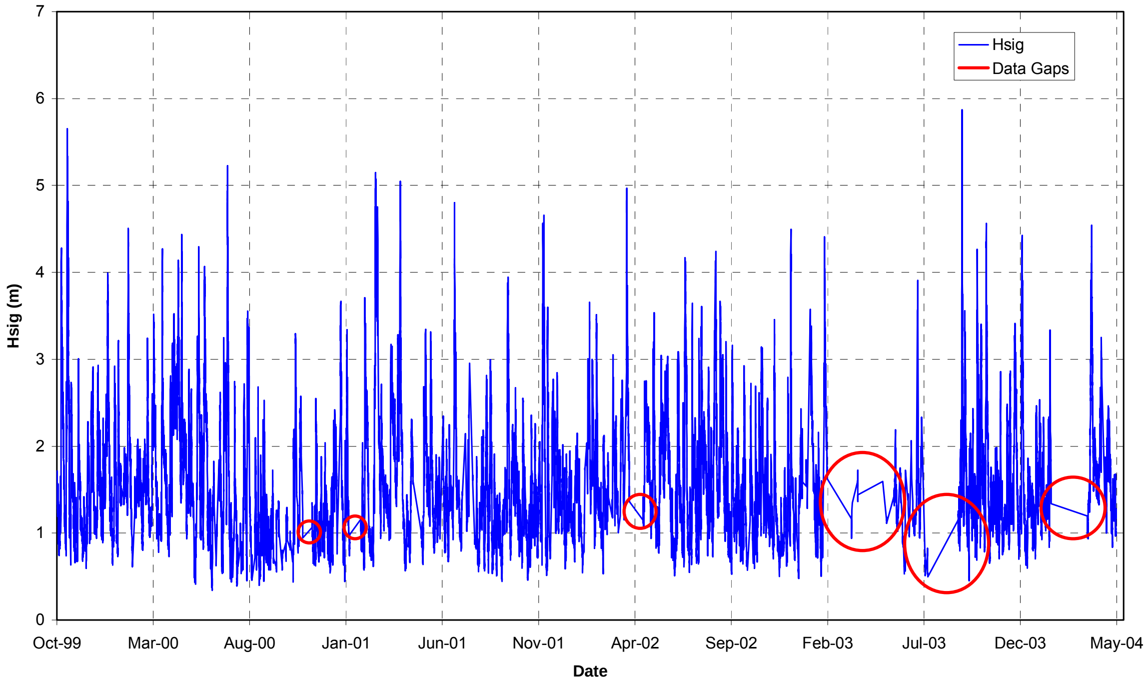


Figure 6-7 Byron Bay Wave Data 1999-2004

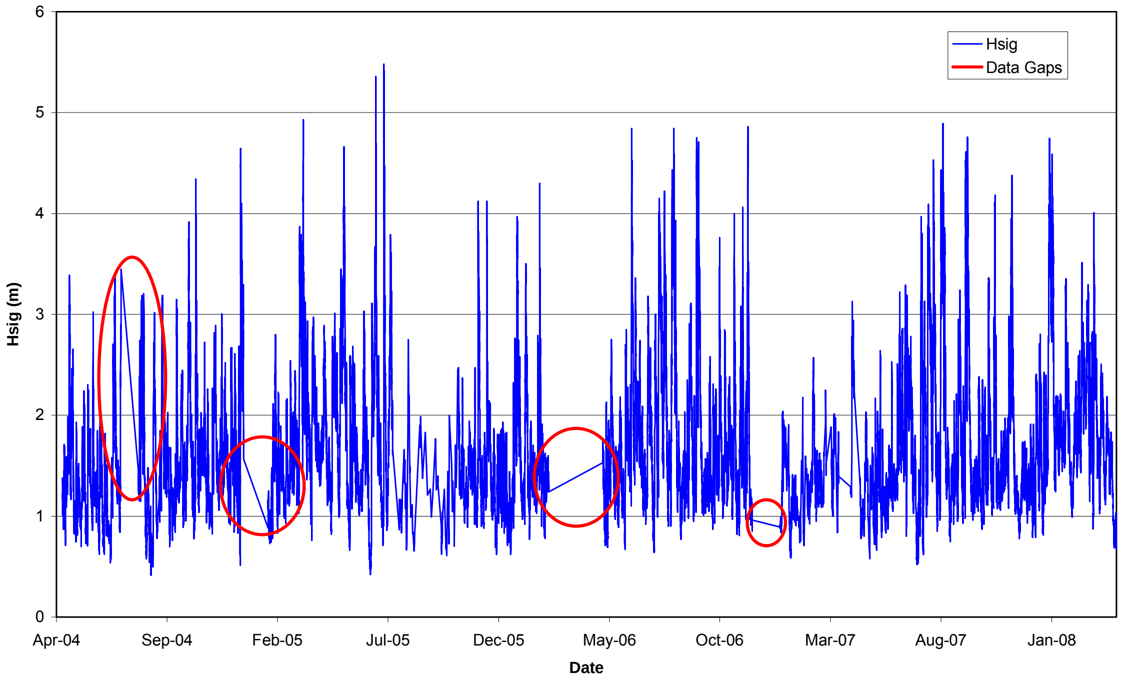


Figure 6-8 Byron Bay Wave Data 2004-2008

Table 6-2 Missing Wave Data - Byron Bay

Byron Bay Buoy	Brisbane Buoy	
Period of Missing Data	**Average Significant Wave Height For Record Period (m)**	**Maximum Significant Wave Height For Record Period (m)**
11/11/2000 -12/1/2000	1.57	2.39
1/23/2001 - 2/14/2001	1.73	5.43
4/4/2002 - 4/27/2002	1.63	3.32
2/6/2003 - 5/24/2003	1.75	3.52
7/13/2003 - 9/1/2003	1.84	3.12
1/21/2004 - 3/18/2004	1.03	6.98
7/19/2004 - 8/13/2004	1.36	3.97
1/1/2005 - 2/7/2005	1.86	3.45
Note: Average Significant Wave Height- Brisbane Buoy (20/01/1997 to 06/03/2006)=1.64m		

Acknowledging that the Byron Bay waverider buoy has periodically been unable to provide data during storm swell events (Hsig>3m), the wave climates (wave heights) for Byron Bay and Brisbane Buoy are very similar. Figure 6-9 shows the wave height exceedance plots for the Byron Bay and Brisbane waverider buoys using hourly data for the following record periods;

Brisbane - 20/01/1997 to 06/03/2006; and

Byron - 26/10/1999 to 03/31/2008.

Comparisons between the directional histograms for each of the locations, shown in Figure 6-10, indicate however, that directionally the Brisbane wave rider buoy experiences far fewer swell events propagating from directions greater than 160 degrees. Geographically, this is due to the sheltering effect of Cape Byron, the most easterly point of Australia. The histogram plots were plotted for the record periods above.

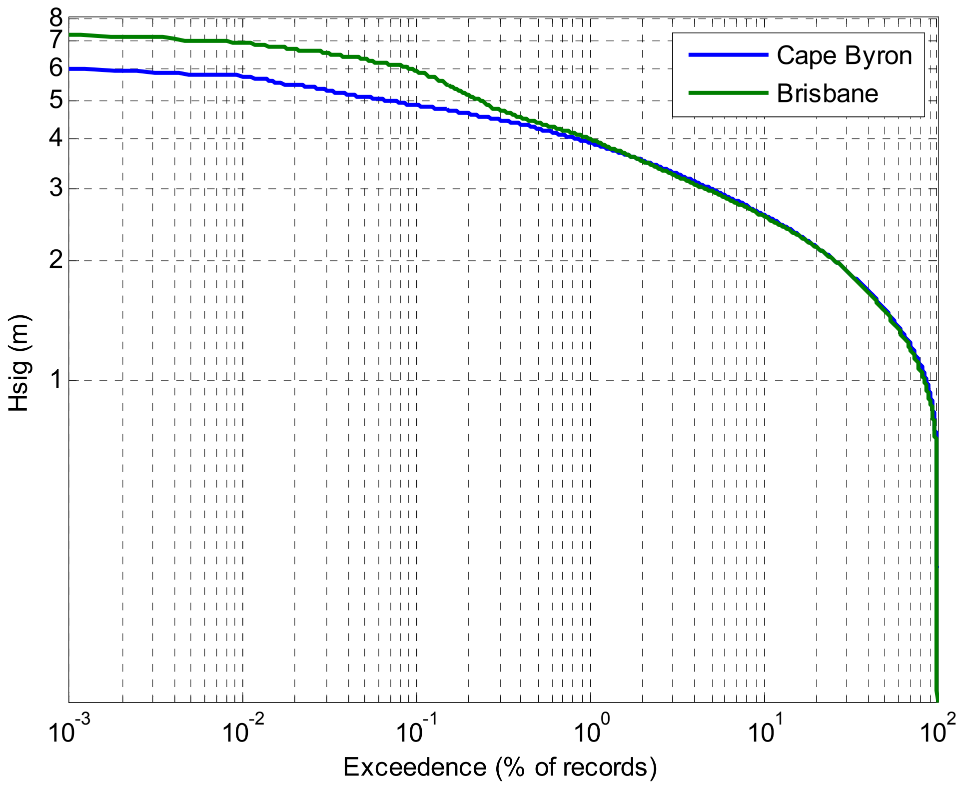


Figure 6-9 Byron Bay/Brisbane- Wave Height Exceedance Plot

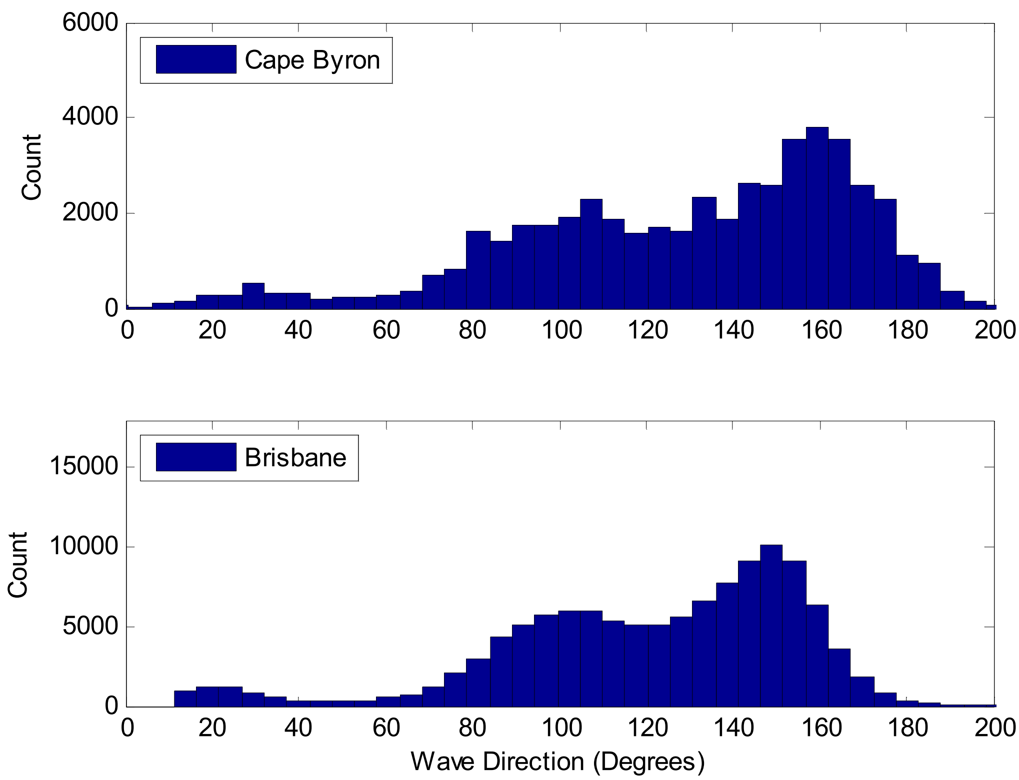


Figure 6-10 Byron Bay/Brisbane- Wave Direction Histogram

To supplement the missing data for Byron Bay, data from the Brisbane waverider buoy was used in conjunction with wave transformation tables derived using SWAN modelling. Methods identical to those described in Section 5.1.1 were used during this assessment. During the data supplement process careful assessment of concurrent data sets representing swell events from the 160 to 180 degrees quadrant was undertaken to ensure the sheltering effect of Cape Byron was represented. This method to fill the gaps in the Byron Bay data was selected in preference to the use of an energy weighted mean of historic data from Byron Bay due to the importance of representing storm events in the sediment transport models. Since most of the gaps in the Byron Bay data represent periods of moderate to large swell events replacing these gaps with an energy weighted mean values would bias towards under predicted wave heights for the synthesised data. This in turn would result in an under prediction of shoreline response in the sediment transport modelling.

Comparisons between recorded data from Byron Bay and modelled data based on Brisbane waverider buoy recordings were undertaken the period between June 2001 to November 2001. This period represents a duration with concurrent datasets exhibiting few gaps for both the Byron Bay and Brisbane datasets.

Acknowledging the distance between the two buoy locations (approximately 150km), and hence, the possibility for spatial variation in swell generation, the validation of the wave transformation model has been shown to adequately represent the wave conditions for Byron Bay. Table 6-3 and Figure 6-11 to Figure 6-12 show the model validation results.

Figure 6-13 to Figure 6-15 show the Byron Bay waverider buoy data including the calculated data supplementing the periods with missing data in the raw data provided by the Manly Hydraulics Laboratory.

Table 6-3 Wave Transformation Model Validation Results- Byron Bay

Comparison Period	Location	Mean Hsig (m)	Hsig Standard Deviation	Mean Direction (Degrees)	Direction Standard Deviation
June 2001 - November 2001	Recorded Wave Data Brisbane Buoy	1.38	0.60	121.8	26.49
June 2001 - November 2001	Recorded Wave Data Byron Bay	1.32	0.60	127.0	45.3
June 2001 - November 2001	Modelled Wave Data Byron Bay	1.38	0.65	126.4	29.5

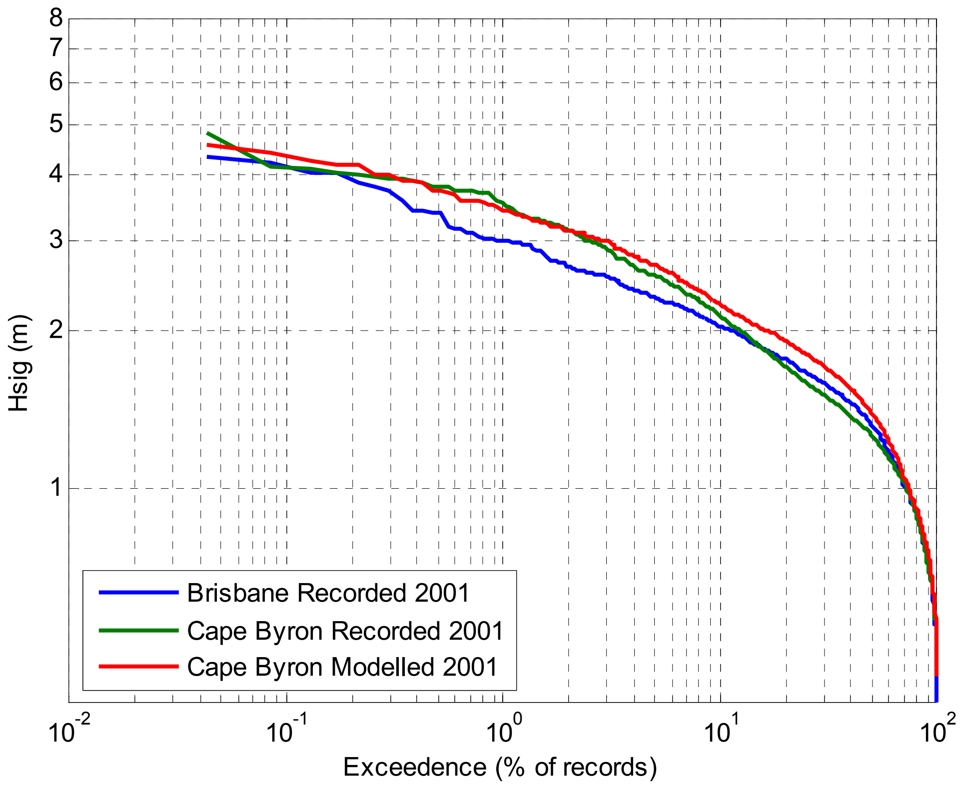


Figure 6-11 Wave Transformation Model Validation- Wave Height Exceedance Byron Bay 2001

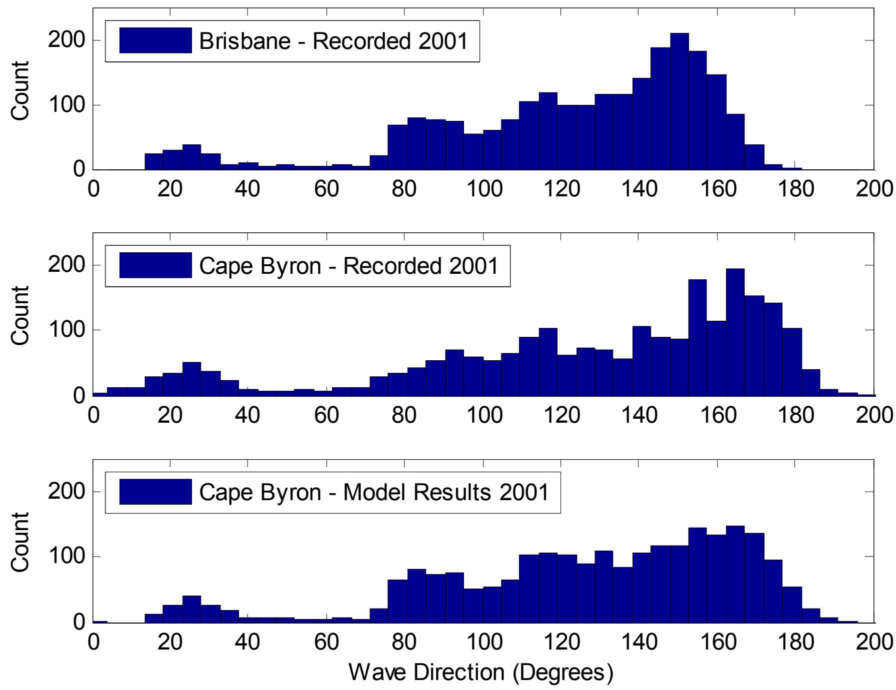


Figure 6-12 Wave Transformation Model Validation- Wave Direction Histogram Byron Bay 2001

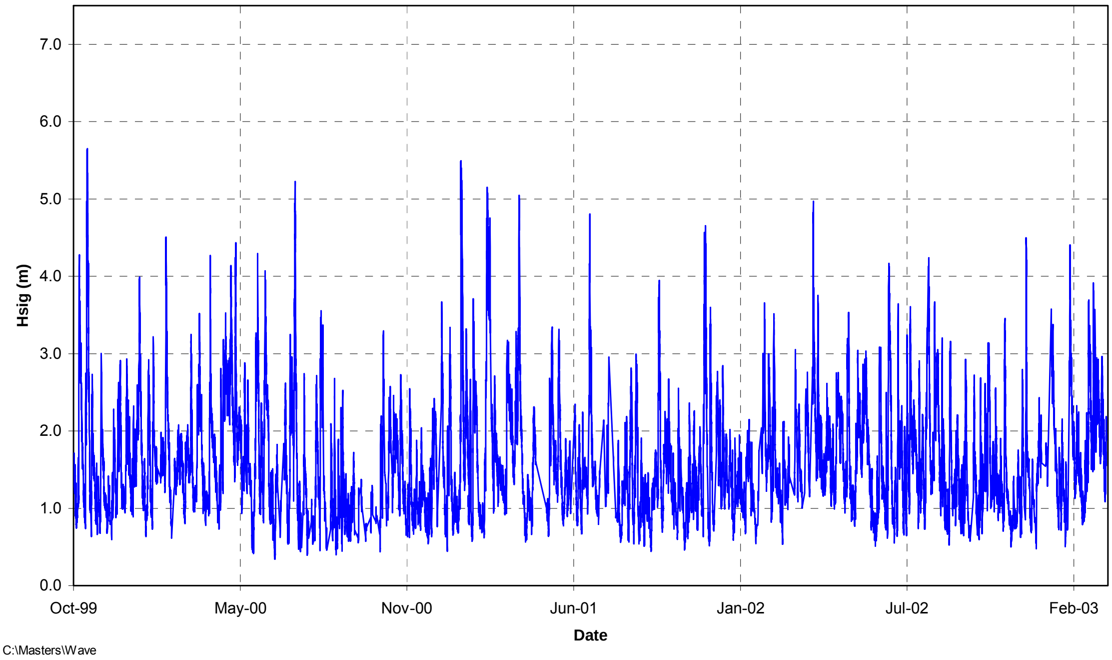


Figure 6-13 Gap Filled Byron Bay Wave Data 1999-2003

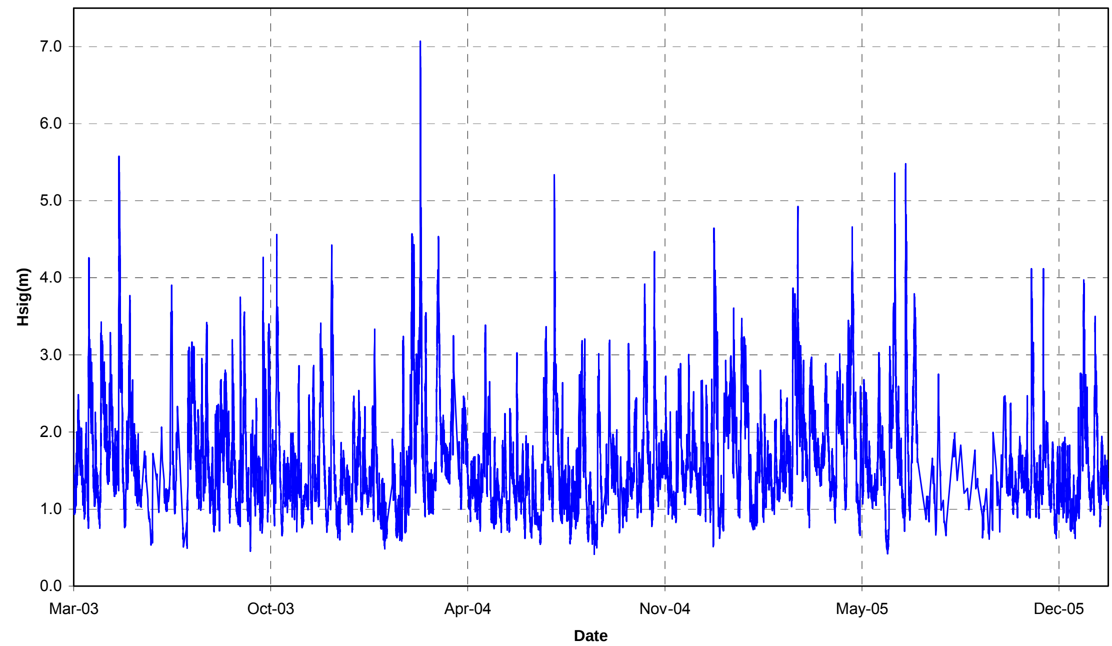


Figure 6-14 Gap Filled Byron Bay Wave Data 2003-2005

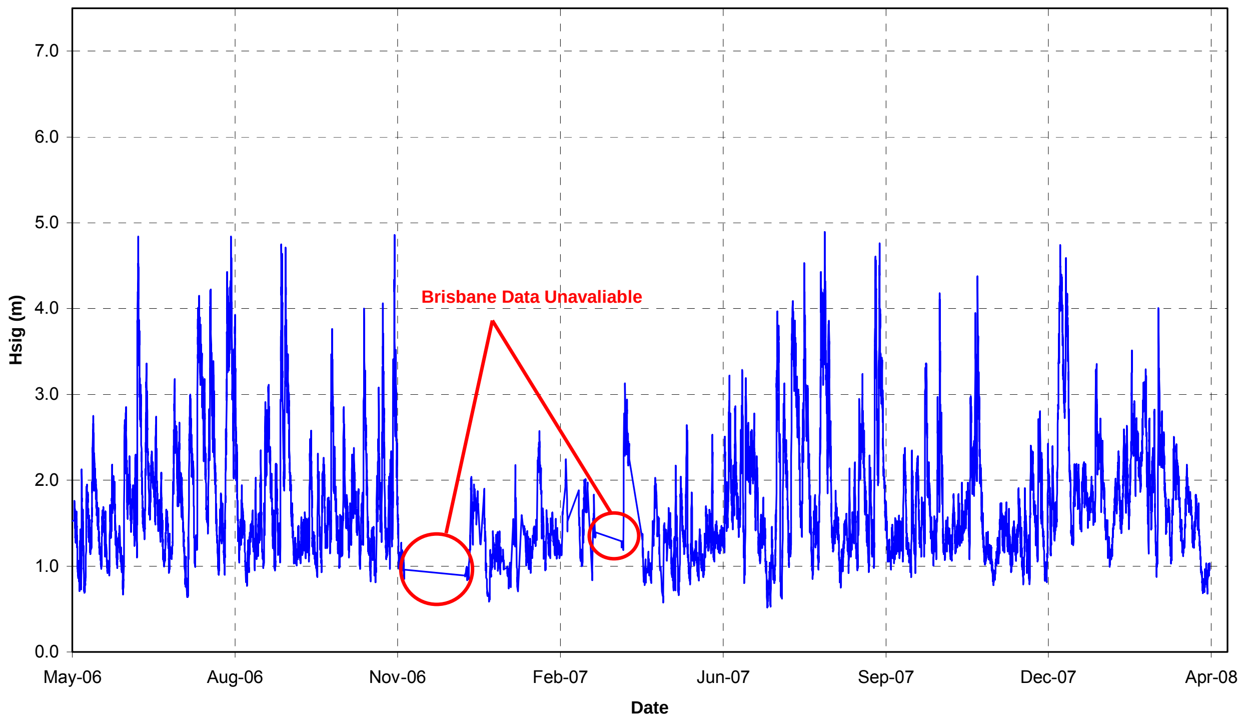


Figure 6-15 Gap Filled Byron Bay Wave Data 2005-2008

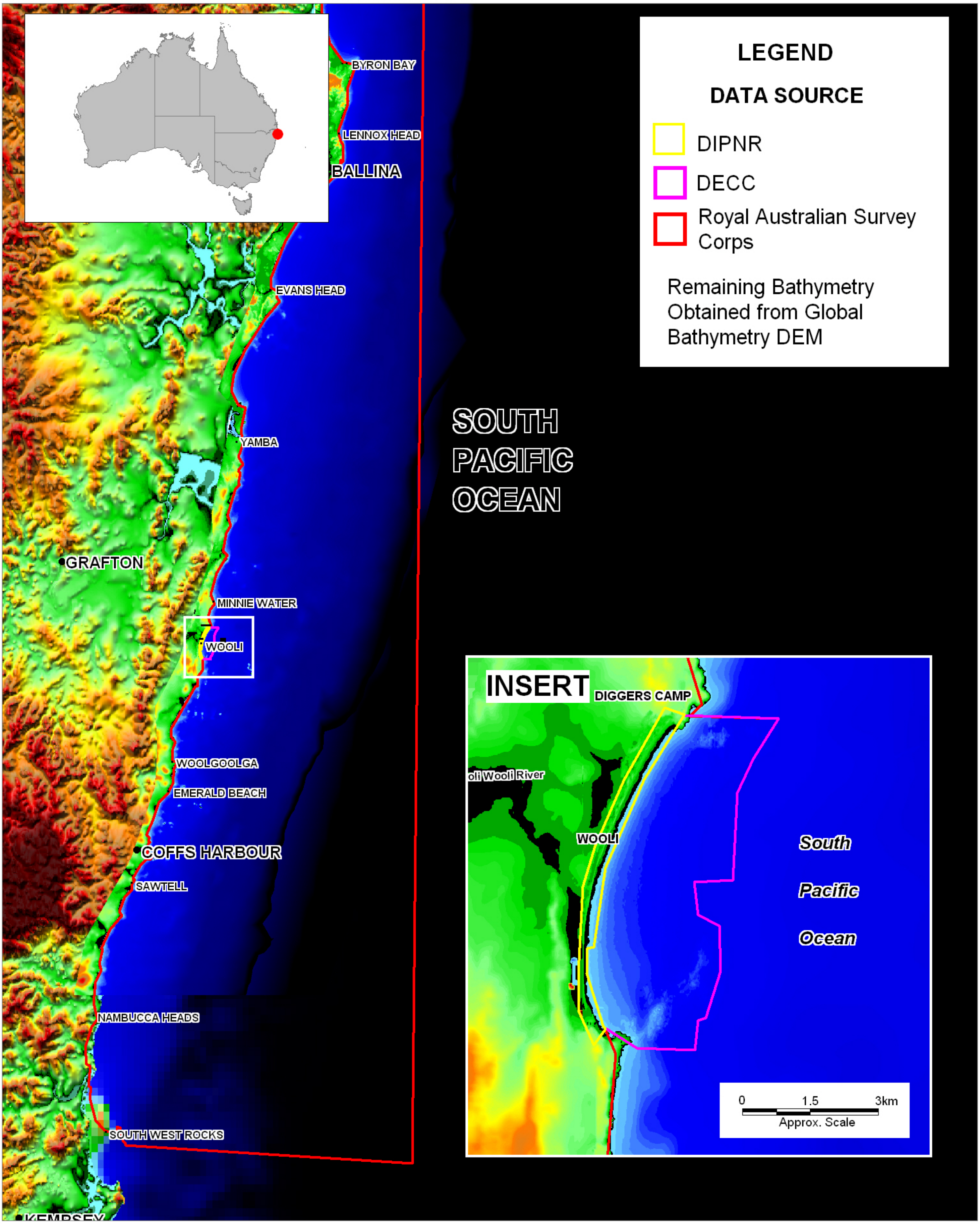
6.1.3 Bathymetry Data

Representation of the bathymetry offshore from Wooli Wooli was obtained from various sources. Figure 6-16 and Table 6-4 outlines the data sources used to define the offshore bathymetry and the preference of usage for each of the datasets.

Table 6-4 Wooli Wooli Bathymetric Data

Bathymetric Data Description	Source	Usage Preference
Wooli Wooli Beach Shoreline/Dunes	NSW Department of Infrastructure Planning and Natural Resources (DIPNR) - Photogrammetry survey (1996)	1
Nearshore	Department of Environment and Climate Change (DECCW) - Hydrographic Survey (October 2007)	2
Offshore Deepwater/Nearshore Bathymetry	Royal Australian Survey Corps - Bathymetric Charts (1959)	3
Offshore Deepwater Bathymetry	Course Global DEM	4

In the nearshore region of the study area, the most recent available bathymetry/shoreline data were collected in 1996. This dataset has been used as the base bathymetry dataset within the sediment transport models discussed in the following sections. In addition, to define the required profile parameters for the cross shore modelling tasks, survey of the nearshore surfzone was completed as part of this study.



Title:

Wooli Wooli Bathymetric Data

Figure: 6-16

Rev: A

N

0 30 60km

Approx. Scale

BMT WBM
www.wbmpl.com.au

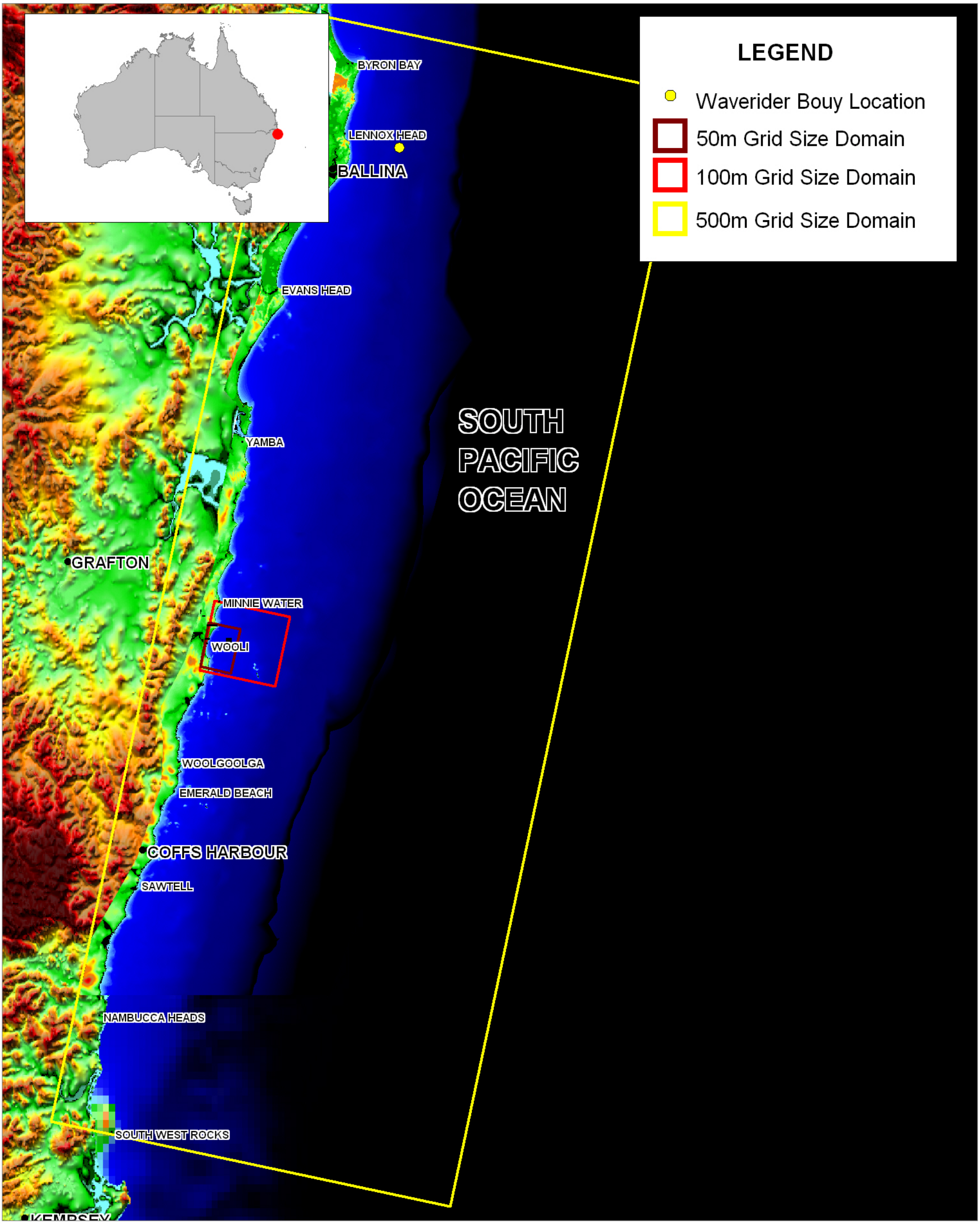
Filepath : I:\B16444_I_BRH Climate Change IAT\DRG\COA_003_080625_WW_Bathymetry.wor

6.1.4 Shoreline Change Modelling

Shoreline change modelling for Wooli Wooli beach has been completed for the historic period between 1/11/1999 and 1/11/2007 and three future climate change scenarios centred on the 2030, 2070 and 2100 planning horizons. Details outlining the model setup, historic assessment verification and climate change assessment are outlined in the following sections.

6.1.4.1 Wave Transformation Modelling

A multiple domain SWAN model has been developed to transform deepwater wave inputs into the nearshore. The developed model utilises three separate domains with grid resolutions of 500m, 100m and 50m respectively, as shown in Figure 6-17. The finer mesh grids are applied in the nearshore to provide the higher resolution results required to drive the sediment transport models developed for Wooli Wooli Beach. No available buoy data were available for offshore from Wooli Wooli to validate the developed wave transformation model. Due to this, identical model calibration parameters used during the development of the Gold Coast wave transformation model were applied to the Wooli Wooli wave transformation model.



Title:
Wave Transformation Model - Wooli Wooli

Figure: 6-17

Rev: A

N

0 30 60km

Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : I:\B16444_I_BRH Climate Change IAT\DRG\I:\B16444_I_BRH Climate Change IAT\DRG\COA_004_080625_WW_SWAN.wor

6.1.4.2 Longshore Transport Modelling

A longshore sediment transport model (LSMOD) was developed to represent the full extent of Wooli Wooli beach. The model was developed based on the calibration parameters developed during the Gold Coast longshore sediment transport analysis. The initial shoreline position used for the modelling was sourced from the most recent available shoreline/dune survey data available for Wooli Wooli. The 1996 NSW Department of Infrastructure Planning and Natural Resources (DIPNR) photogrammetry survey was used for this purpose.

Using the calibration parameters applied at the Gold Coast, a K value of 0.14 was used. A nested model was also applied, though due to unique bathymetric features (reefs) offshore from Wilsons Headland in the north and the Wooli Wooli River in the south, a modified nesting approach was developed. For the Gold Coast model, nesting of the wave transformation model was applied at 15m of water depth. Unfortunately, as shown in Figure 6-18 the offshore reefs mentioned above are located landward of the 15m depth contour. If the nested model was to be applied to the 15m depth contour and wave transformation resulting from these two reefs would not be represented in the LSMOD model, since LSMOD defines the bathymetry landward of the nesting location using parallel contour assumptions based on the shoreline orientation. Figure 6-18 shows the 15m depth contour offshore from Wooli.

To resolve this issue, the wave transformation model results were applied at the 5m-depth contour. Linear wave theory was used to back calculate the equivalent wave height and directions at the 10m depth contour. This back calculation was done to ensure the nearshore wave nesting tables represented wave conditions seaward of the wave break point for all wave conditions. The calculated wave transformation tables representing wave conditions at the 10m-depth contour were used to drive the developed longshore transport model for Wooli Wooli Beach.

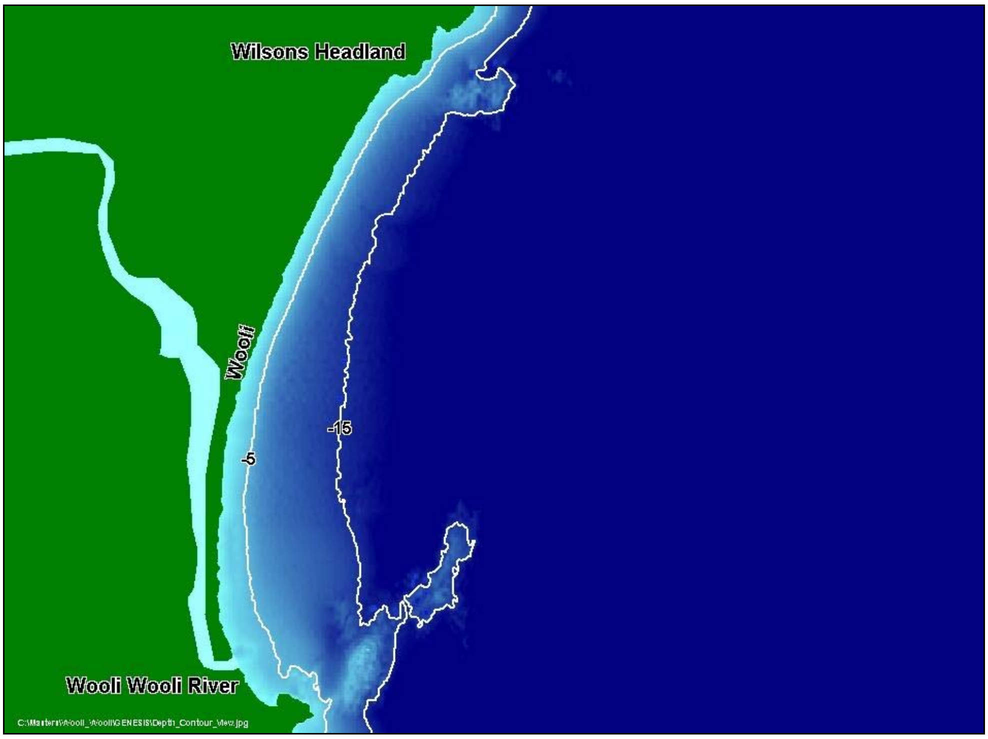


Figure 6-18 Wooli Wooli Model Set-up – Offshore Bathymetry

The developed nested model was used to calculate longshore sediment transport for the available gap filled waverider buoy data from Byron Bay. The model was run for the period from 1/11/1999 till the 1/11/2007. Figure 6-19 shows the modelled initial shoreline position, based on the 1996 shoreline survey data. Figure 6-20 shows the shoreline change relative to the initial shoreline position for the developed longshore model. A positive value represents shoreline accretion and a negative value represents shoreline erosion. Although, no recorded survey data is available to validate the developed longshore model, the model results confirm that the baseline model shoreline for the current situation is in a relatively stable orientation. This trend has been verified based on correspondence with local residents and Clarence Valley Council officers. For reporting purposes, the maximum modelled setback (land most shoreline position) and the final shoreline position at the end of the simulation have been reported.

Along the main section of Wooli Wooli Beach the modelled shoreline change due to longshore transport processes has generally been within the limits of ±10.0m. Adjacent to the northern and southern headlands of the beach, increased variation in shoreline position is experienced. Analysis of the timeseries results for the model simulation indicate that the increased shoreline erosion in these locations predominately occur during storm events exhibiting an easterly wave direction. During these events, diverging longshore transport patterns occur adjacent to the headland control, primarily due to the beach orientation relative to the incident wave direction. Adjacent to the south of the headland, sediment is transported in a southerly direction, whereas north of the headland the longshore sediment transport is predominantly to the north. During major swell events this longshore sediment transport pattern may cause significant longshore transport driven shoreline erosion adjacent to the headland controls. Due to the sheltering effect of the reefs offshore from the southern headland, this erosion occurs to a lesser magnitude along the southern section of Wooli Wooli beach. Along the northern section of Wooli Wooli Beach the most landward shoreline position was calculated to be approximately -30.0m due to longshore processes only.

Figure 6-21 shows the corresponding calculated annual net longshore sediment transport rate for the modelled period. Figure 6-22 shows a time series of the mean sediment transport rates for Wooli Wooli Beach over the duration of the assessed period. These reported values represent mean values averaged over the entire length of the beach. In both figures, the negative sediment transport values denote that the sediment transport is primarily in a northerly direction. Averaging the results shown in Figure 6-21, the mean annual longshore transport for Wooli Wooli Beach is approximately 210,000m^3/year to the north. This estimate is inline with decreasing trend in annual net transport rates heading south from Byron Bay document by Patterson (2007a).

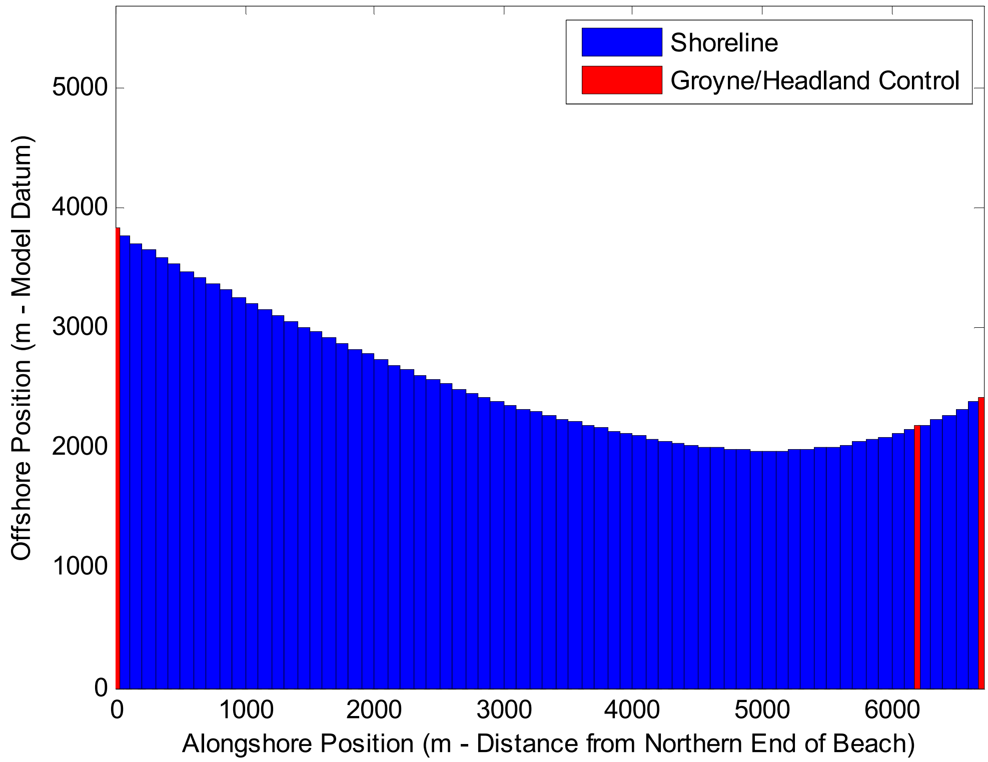


Figure 6-19 Wooli Wooli Longshore Transport Model Domain

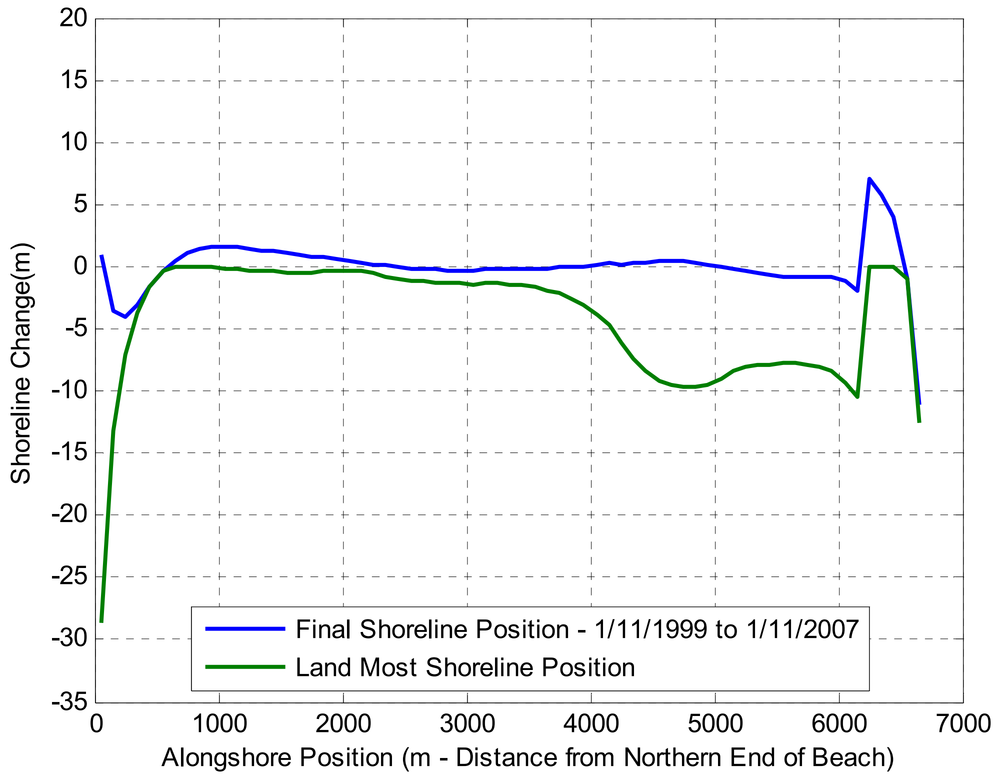


Figure 6-20 Longshore Transport Model Shoreline Change Results

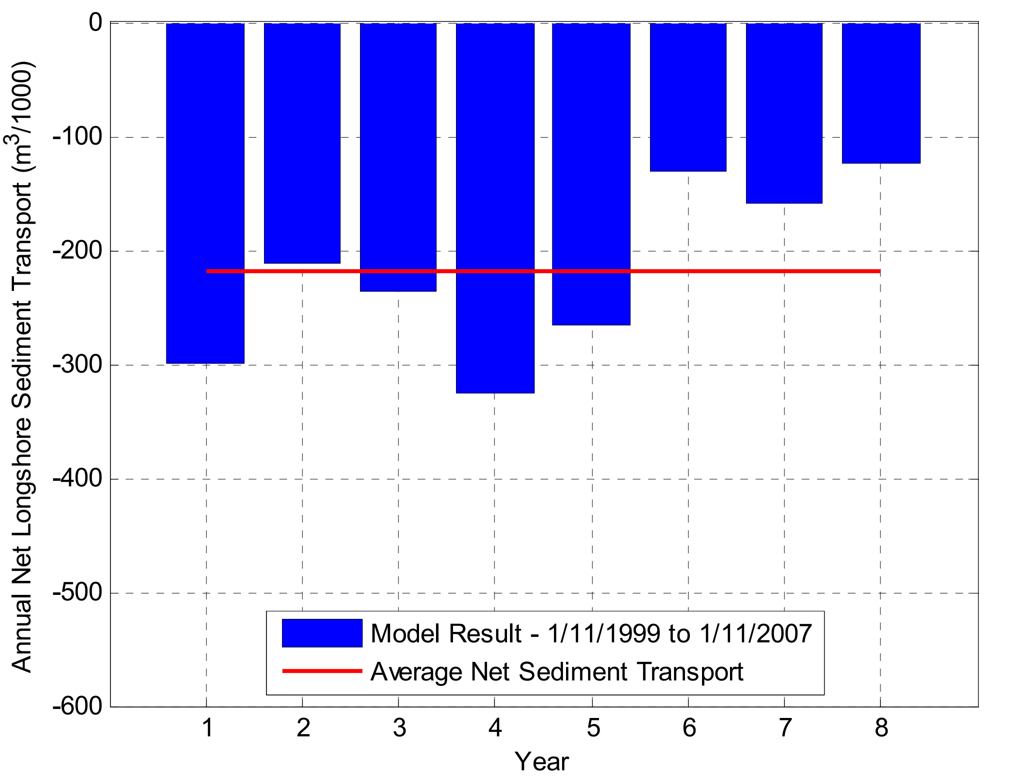


Figure 6-21 Longshore Transport Model Net Sediment Transport Results

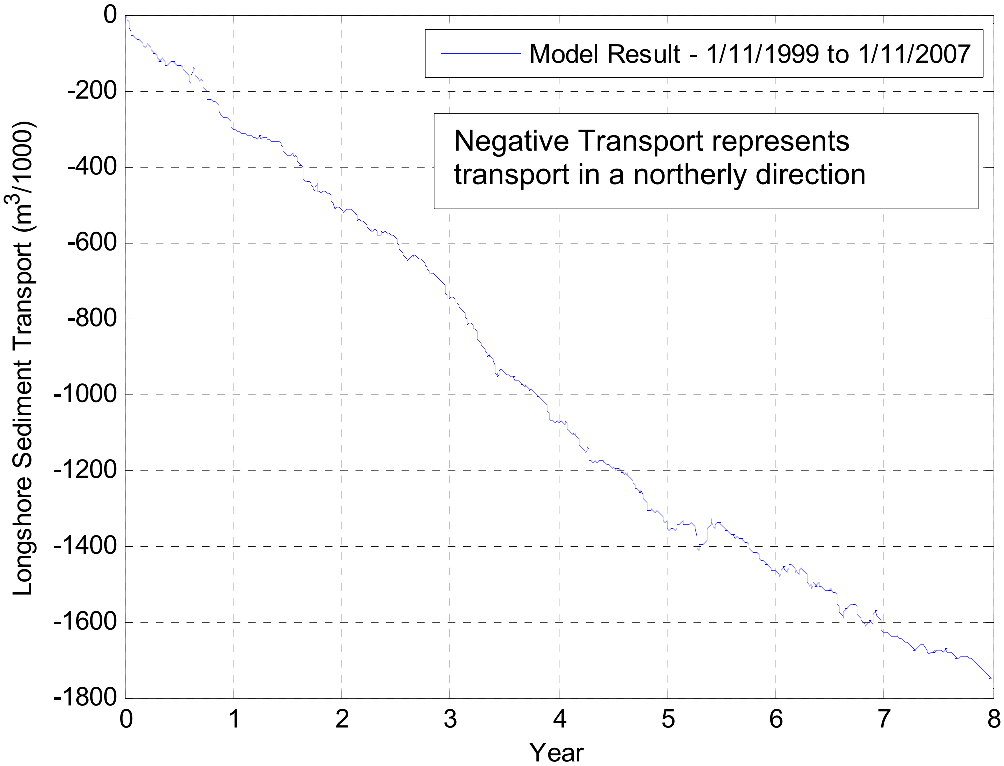


Figure 6-22 Longshore Transport Model Timeseries Results

6.1.4.3 Cross-Shore Profile Modelling

Cross-shore modelling was undertaken for 5 locations along Wooli Wooli Beach, shown in Figure 6-24, for the period from 1/11/1999 till the 1/11/2007 using the gap-filled Byron Bay waverider buoy data. Recorded water level data from the Yamba offshore recording station, supplied by the Manly Hydraulic Laboratory, was used to represent the historic tide and surge levels for the modelled period. The geometric cross-shore model described in Section 4.3.2 was used for the assessment.

To represent the complex wave transformation processes offshore from Wooli Wooli Beach, the methodology discussed in Section 6.1.4.2, using SWAN to transform deepwater waves to 5m of water depth before back calculating wave transformation tables to 10m of water depth (offshore from the maximum breaker depth) using linear theory has been applied. From 10m of water depth the input wave forcings are refracted and shoaled to the depth of breaking using linear theory.

The model rate constant parameters used during the cross shore modelling of Wooli Wooli beach were based on the calibration parameters defined during the cross-shore model validation procedure undertaken using data from the Gold Coast. The sediment and profile parameters used during the cross-shore modelling of Wooli Wooli beach was obtained from site visit data and extracted from the nearshore hydrographic and shoreline photogrammetry surveys provided by the DECCW and DIPNR respectively. Figure 6-23 shows an example of the initial profile definition for a location midway along Wooli Beach, including the various profile slope components (introduced in Section 4.3.2), compared with survey data mentioned above.

Table 6-5 shows the model parameters used during the cross-shore modelling of Wooli Wooli Beach.

Table 6-5 Cross-Shore Model Calibration Parameters –Wooli Wooli

Sediment/Profile Parameters	**Rate Constant Parameters**
Grain Size (GS) = 0.3mm	Erosion Rate Constant (ke) = $2e^{-6}$
Deepwater Slope (m_2) = 100m/m	Accretion Rate Constant (ka) = $8e^{-9}$
Transition Slope (m_1) = 35m/m	
Dune Slope (m_0) = 1m/m	
Dune Height (B) = 5m (L5) to 10m (L2)	
Sediment Scale Parameter (A) = 0.125	
Bar Extent (h_b) =0.7	

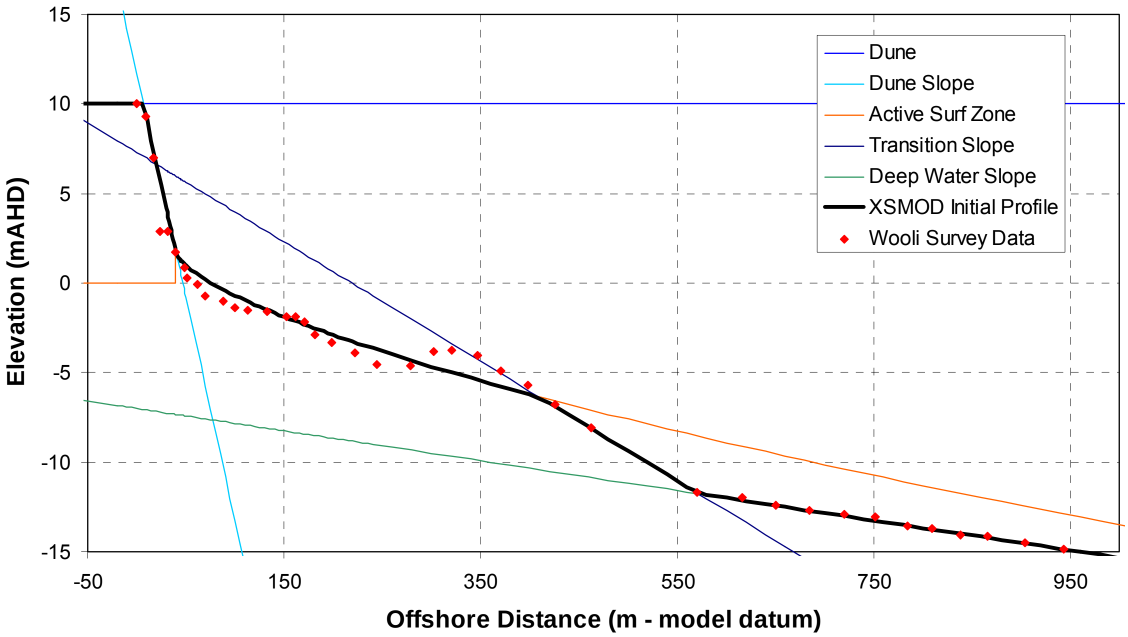


Figure 6-23 XSMOD Initial Profile

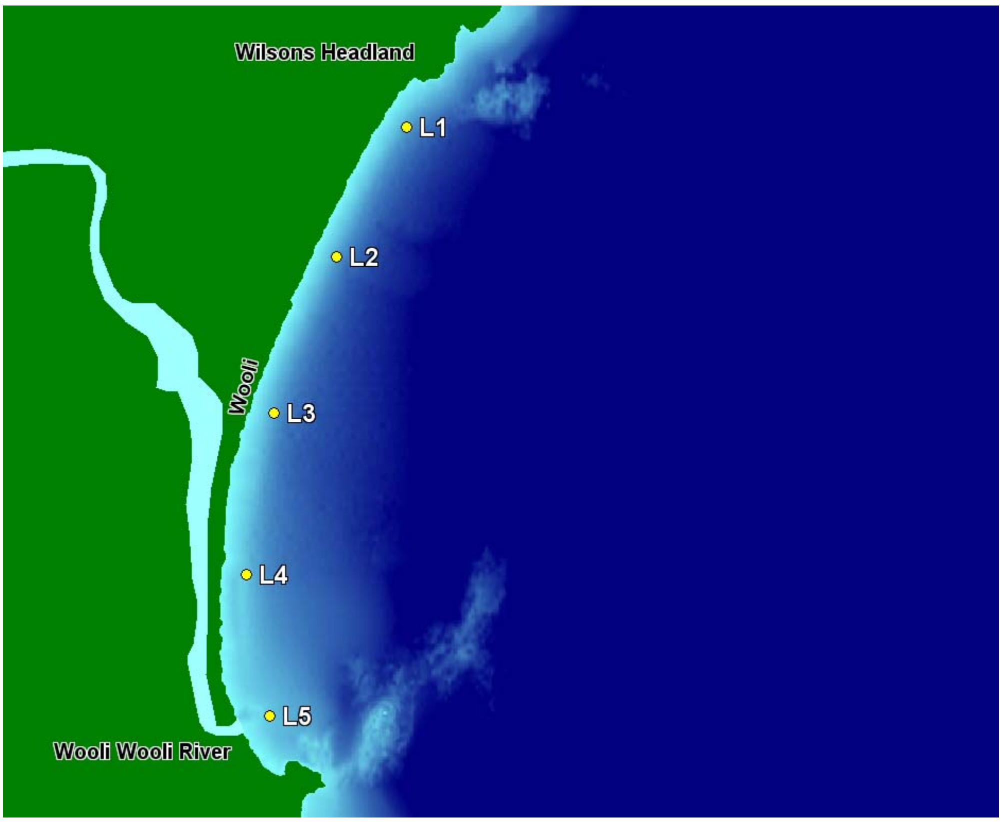


Figure 6-24 Cross-Shore Model Assessment Locations – Wooli Wooli

Figure 6-25 and Figure 6-26 show the modelled cross-shore profile response, in terms of dune volume and shoreline position, from 1/11/1999 to 1/11/2007 for three locations along Wooli Wooli Beach. As outlined in Section 6.1.4, the shoreline change results represent a shift in the dune face, not the beach berm.

The cross-shore modelling results indicate that during the modelled period the greatest volume of sediment eroded from the beach dune ranged from approximately 90m^3/m along the southern section of Wooli Wooli Beach (L5) and 150 m^3/m along the northern section beach (L1). This change in dune volume equates to a variation in dune face of approximately a 10m in the south and a 15m in the north.

From the variation in alongshore results shown in Figure 6-25 and Figure 6-26 (i.e. comparing L1 and L5 results) the following cross shore response behaviours have been identified:

1. Due to wave refraction and sheltering in the vicinity of the headland to the southern end of Wooli Wooli beach, storm events producing waves from the southern quadrant resulted in greater cross-shore erosion along the northern section of Wooli Wooli beach than along the southern section. Similarly, storm events producing waves from the northern quadrant result in greater cross-shore erosion along the southern stretch of Wooli Wooli beach.

2. Dune height progressively increases moving north along Wooli Beach. The model results below show that for equivalent erosion setback distances, sections of Wooli Wooli beach exhibiting lower dune heights (L5) experience reduced dune erosion volumes.

In addition to the above listed cross-shore response behaviour, analysis of individual storm events which occur during the modelled period has also highlighted the significant relationship between coincident storm waves with elevated ocean levels. During the modelled period, numerous erosion events occurred, with the most significant erosion period occurring during March of 2001 and March of 2004. During these two erosion periods, resulting from multiple erosion events, shoreline setbacks ranged from 5 to 15 metres along Wooli Wooli Beach.

Assessment of the input wave conditions have identified various periods of extreme waves which did not result in large erosion volumes. In particular, during the storm event of early May 2003, which exhibits the second largest significant wave heights for the modelled period the model results only show between 25 and 50 m^3/m of dune erosion along Wooli Wooli beach. Inspection of the recorded offshore water levels indicates that this event occurred over a neap tide cycle where the ocean water levels were relatively low, compared with the spring tide cycle. Combined with a relatively small storm surge, the May 2003 event did not result in major erosion of the dune face.

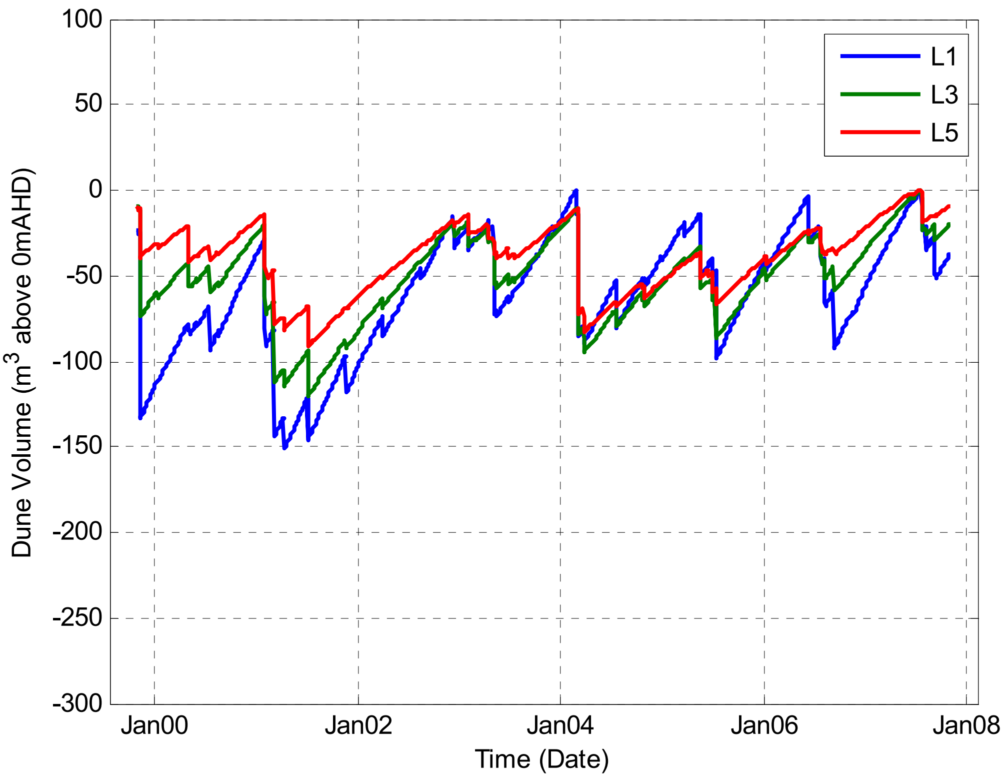


Figure 6-25 Cross-Shore Model Dune Volume Results 1999-2008 – Selected Locations

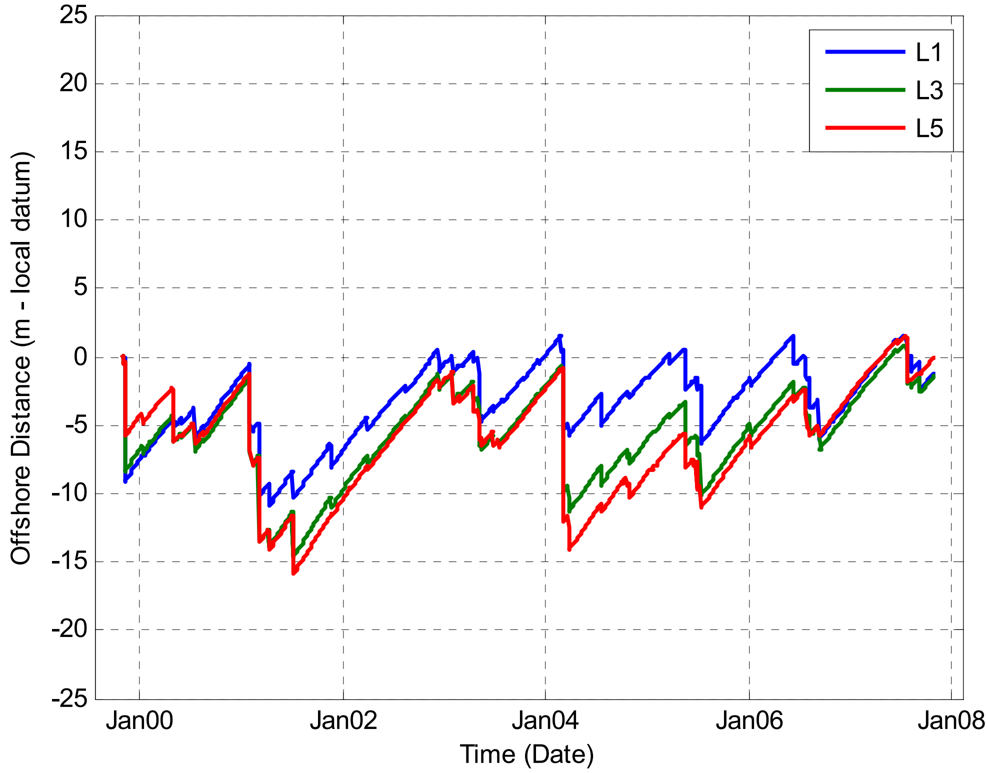


Figure 6-26 Cross-Shore Model Shoreline Position Results 1999-2008 – Selected Locations

As a sensitivity test, the May 2003 event has been replaced with an event representative of the 1% AEP wave height and storm surge coinciding with a mean high water spring tide. To create this design event, the May 2003 event was modified to include the follow wave height and water level conditions:

- Hsig = 7.75m (Allen and Callaghan, 2001);
- Peak Wave Height Duration = 5 hours;
- Direction = East (selected, as it results in the greatest exposure for the majority of Wooli Wooli Beach);
- Surge = 0.67m (McInnes *et al.*, 2007); and
- Tide Timing = Coinciding with a Mean High Water Spring tide (MHWS).

The constructed event is shown below in Figure 6-27.

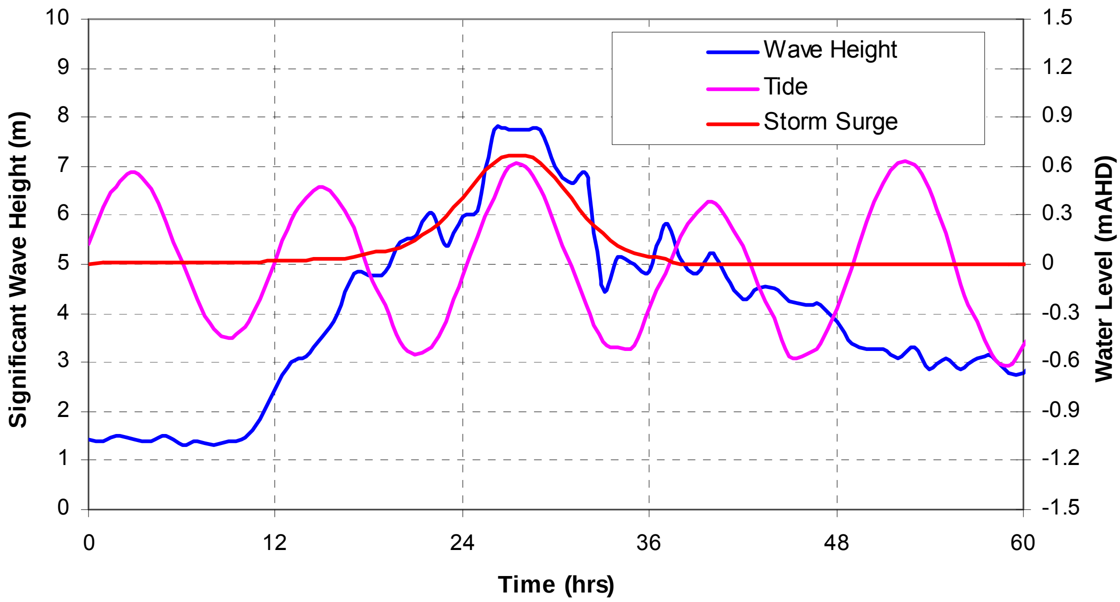


Figure 6-27 Wooli Wooli 1%AEP Storm Event

The sensitivity assessment results, shown in Figure 6-28 and Figure 6-29, show that replacing the May 2003 event with an equivalent 1%AEP event results in increased shoreline erosion. Shoreline setback of approximately 20m is modelled for L5, the southern end of Wooli Beach.

As an isolated event, the 1% AEP event results in the greatest shoreline erosion for the modelled period. This erosion volume/setback is however approximately matched at different locations along Wooli Wooli beach with the shoreline response during either the March 2001 or March 2004 erosion periods. These results indicate that multiple smaller magnitude events in quick succession are likely to result in increased erosion, equivalent to a single event of significantly greater magnitude.

Comparing the 1% AEP event results with the historic modelled period shown in Figure 6-25 and Figure 6-26, by January of 2006 the cross-shore profile has recovered from the modelled 1% AEP storm event. As such, from January of 2006 through to the end of the model simulation, both the historic shoreline assessment and the storm erosion sensitivity results exhibit similar dune volume and shoreline response trends.

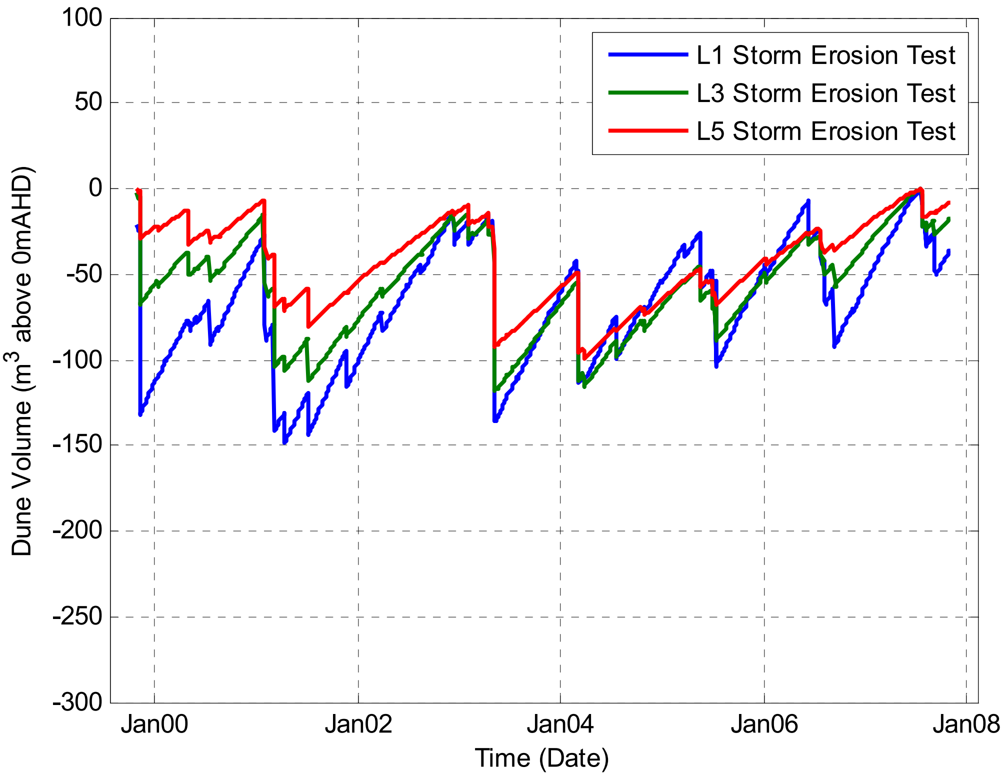


Figure 6-28 Cross-Shore Model Dune Volume Results - 1% AEP Event Sensitivity Test

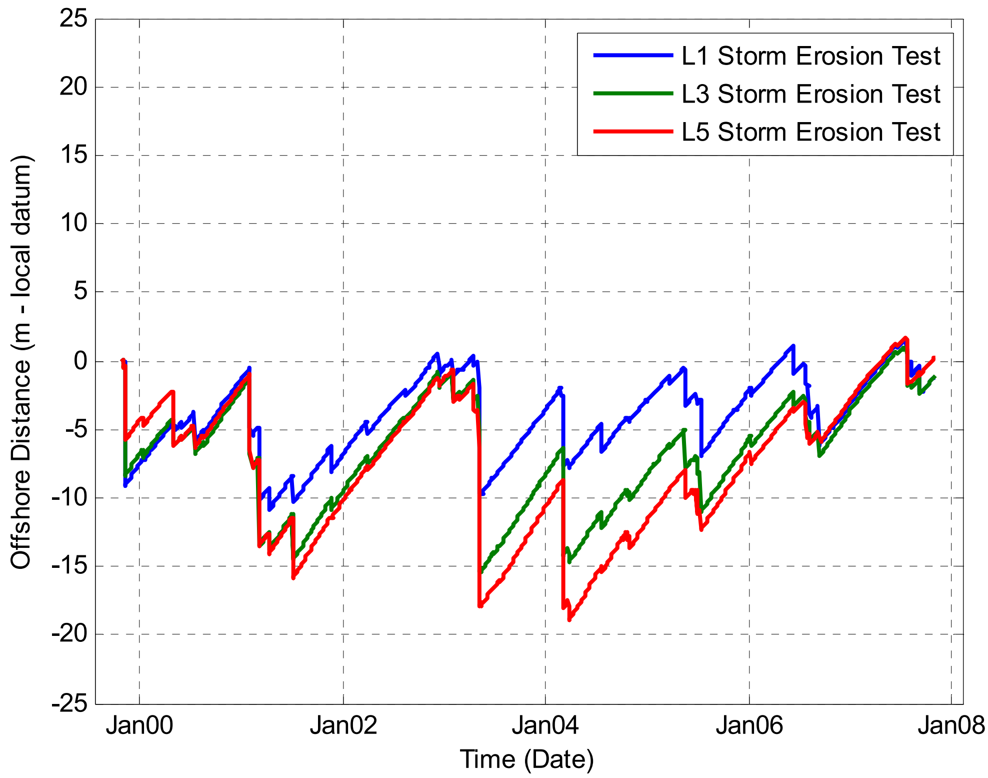


Figure 6-29 Cross-Shore Model Shoreline Position Results - 1% AEP Event Sensitivity Test

6.1.5 Combined Cross-Shore / Longshore Model Results

Using the procedure outlined in Section 4.4 the combined impact of the cross-shore and longshore sediment transport on Wooli Wooli beach has been modelled from 1/11/1999 till the 1/11/2007. The result of this assessment is shown in Figure 6-30. Overall the model results show that the shoreline position has been relatively stable for the period from 1/11/1999 till the 1/11/2007. Along the main section of Wooli Wooli Beach the shoreline change has approximately been within the limits of ±15.0m. Adjacent to the northern and southern headlands of the beach, increased variation in shoreline position is experienced. The reason for this increased variation is due primarily to longshore sediment transport gradients, as discussed in Section 6.1.4.2.

Unfortunately there is no recorded wave data available to create a model corresponding to the historic survey datasets for Wooli Wooli beach. However, based on resident comments regarding shoreline change since 1999, the modelled estimate of change in shoreline position are within realistic limits.

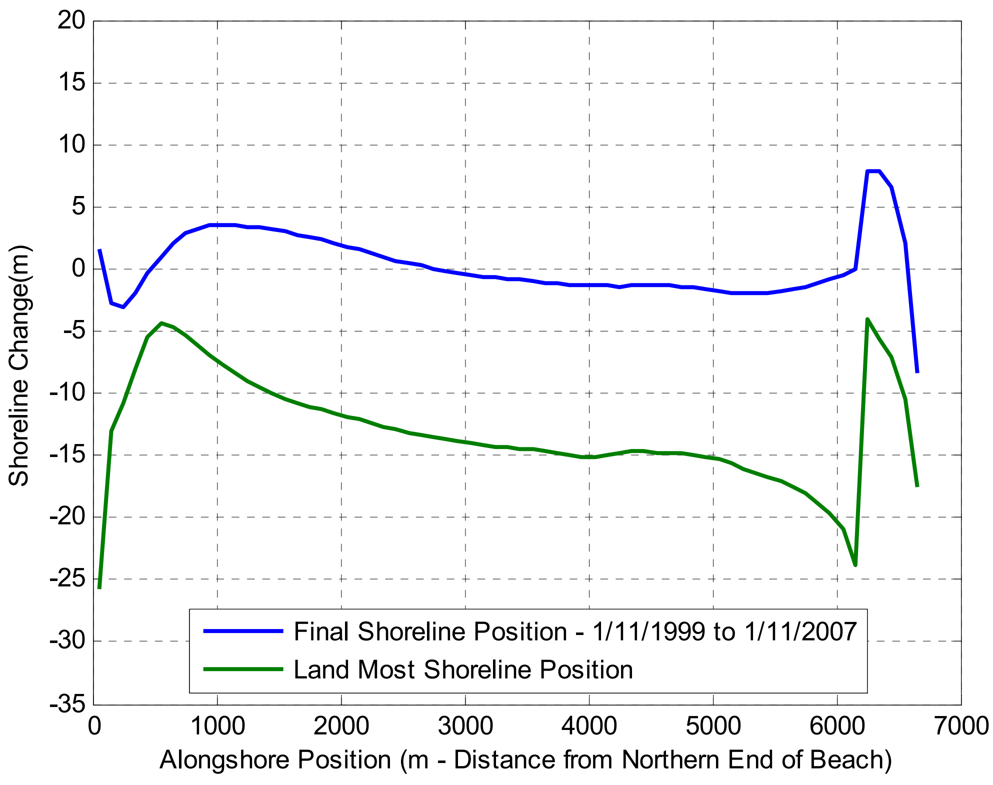


Figure 6-30 Combined Longshore/Cross-shore Model Results

6.1.6 Climate Change Impacts at Wooli Wooli

6.1.6.1 Climate Change Analysis

An assessment of the shoreline response to climate change has been undertaken by applying a scenario based analysis. The climate change scenario assessment was undertaken using the historical wave conditions between 1/11/1999 and 1/11/2007 looped to synthesise 100 years worth of forcing data. To provide results which give an appropriate assessment of erosion hazard for each of the assessed planning horizons, the 1% AEP event used during cross-shore model storm erosion sensitivity assessment has also been embedded within the synthesised 100 years of wave/water level boundary condition data. The 1% AEP storm was included in the wave time series at the reported 2030, 2070 and 2100 planning horizons.

Using the 100 year wave climate timeseries three climate change scenarios were assessed. These scenarios included the CSIRO CCM2 and CCM3 climate change impact projections (McInnes et al., 2007) and also a sea level rise only scenario. The sea level rise only scenario was included to assess the relative impacts of the changes in wave climate documented by the CSIRO, compared with sea level rise only.

To update the climate change scenario input boundary conditions, water level and wave conditions were linearly perturbed over time based on the values documented in Table 6-6. The values listed for 2100 in Table 6-6 were calculated by linearly extrapolating the 2030 and 2070 CSIRO predictions.

Table 6-6 Wooli Wooli Climate Change Forcings

Assessment Scenario		Climate Change Parameter		2000	2030	2070	2100 [b]
Existing Wave Climate (Exg)		Sea Level (m) [a]		0	0	0	0
		Storm Wave Height (m) [c]	NE	0	0	0	0
			E	0	0	0	0
			SE	0	0	0	0
			S	0	0	0	0
		Swell Direction (degrees)		0	0	0	0
		Swell Wave Height (m)		0	0	0	0
Climate Change Scenarios	SLR (Sea Level Rise Only)	Sea Level (m) [a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	0	0	0
			E	0	0	0	0
			SE	0	0	0	0
			S	0	0	0	0
		Swell Direction (degrees)		0	0	0	0
		Swell Wave Height (m)		0	0	0	0
	CCM2	Sea Level (m) [a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	-0.10	+0.20	+0.43
			E	0	-0.10	+0.10	+0.25
			SE	0	+0.30	-0.10	-0.40
			S	0	+0.10	-0.10	-0.25
		Swell Direction (degrees)		0	-3.10	-3.30	-3.45
		Swell Wave Height (m)		0	0	-0.10	-0.18
	CCM3	Sea Level (m) [a]		0	+0.27	+0.60	+0.90
		Storm Wave Height (m) [c]	NE	0	+0.20	+0.40	+0.55
			E	0	+0.10	0	-0.08
			SE	0	-0.10	0	+0.08
			S	0	-0.10	-0.10	-0.1
		Swell Direction (degrees)		0	+0.60	-1.30	-2.73
		Swell Wave Height (m)		0	0	+0.10	+0.18

[a] Sea Level Rise = NSW DECCW sea level rise benchmark (DECCW, 2009)

[b] Value extrapolated from 2030 and 2070 CSIRO prediction (McInnes *et al.*, 2007)

[c] Applied to events exhibiting wave heights greater than 3m Hsig

6.1.6.2 Climate Change Assessment Results

The climate change impact assessment results documenting the Most Landward Shoreline Position (MLSP) modelled for the scenarios listed above are shown in Table 6-7 and Figure 6-31 to Figure 6-33. Figure 6-34 overlays the maximum modelled MLSP from the three assessed climate change scenarios on aerial photography from Wooli Wooli Beach, centred on the 2030, 2070 and 2100 planning horizon. As such, the results in Figure 6-34 do not correspond to a specific climate change scenario.

As shown in the model result, it is evident that, consistent with the behaviour under existing conditions, the projected shoreline response to climate change is not uniform along the beach length. In particular the shoreline recession along the southern portion of the beach is significantly greater than in the north. This is due predominantly to downdrift erosion associated with the groyne/headland control at the southern end which is likely to occur as a result of shoreline recession associated with sea level rise. This process is illustrated in detail in Figure 3-30.

Summarising the results shown in Figure 6-31 to Figure 6-33:

- Centred on 2030 (Figure 6-31), the Sea Level Rise Only and CCM3 results are almost identical. Approximately 11m of climate change related (Climate Change Scenario minus Existing Wave Climate Scenario) shoreline recession is predicted. For the CCM2 scenario, shoreline recession is projected to be approximately 7m greater than the Sea Level Rise Only and CCM3 scenarios. This is directly related to the increased storm wave heights from the SE quadrant, which is the dominant direction of incoming storm waves in the Wooli Wooli region.

- Centred on 2070 (Figure 6-32), the CCM2 and CCM3 results exhibit a similar shoreline response, varying from the Sea Level Rise Only scenario due to the altered wave climate. Similar to the 2030 results the shoreline response results indicate increased shoreline recession at the southern end of Wooli Wooli beach resulting from climate change. The modelled climate change contribution to shoreline recession in this location is approximately 30m.

- For 2100 (Figure 6-33), the difference between the three climate change scenario results increase as the predicted wave climate changes also increase. As per the trend in results for 2030 and 2070, the greatest shoreline recession resulting from climate change is found at the southern end of Wooli Wooli beach, where up to approximately 46.4m of shoreline recession is projected to occur due to climate change. Acknowledging the variation in results between the three climate change scenarios, particularly the increased wave heights from east to north east tropical cyclone sector, the projected changes in sea level are likely to dominate the shoreline response trends compared to the changes in wave climate.

Table 6-7 Wooli Wooli Climate Change Impact Results

Result	*Scenario*	Planning Horizon		
		2030	*2070*	*2100*
Maximum Shoreline Change (MLSP)	EXG	-36.1	-36.1	-36.1
	SLR Only	-47.0	-62.2	-73.4
	CCM2	**-53.4**	-63.8	-79.5
	CCM3	-47.6	**-67.1**	**-82.5**
Maximum Climate Change Related Shoreline Recession (Climate Change Scenario minus Existing Case Scenario Result)	SLR Only	-10.9	-26.1	-37.3
	CCM2	**-17.3**	-27.7	-43.4
	CCM3	-11.5	**-31.0**	**-46.4**

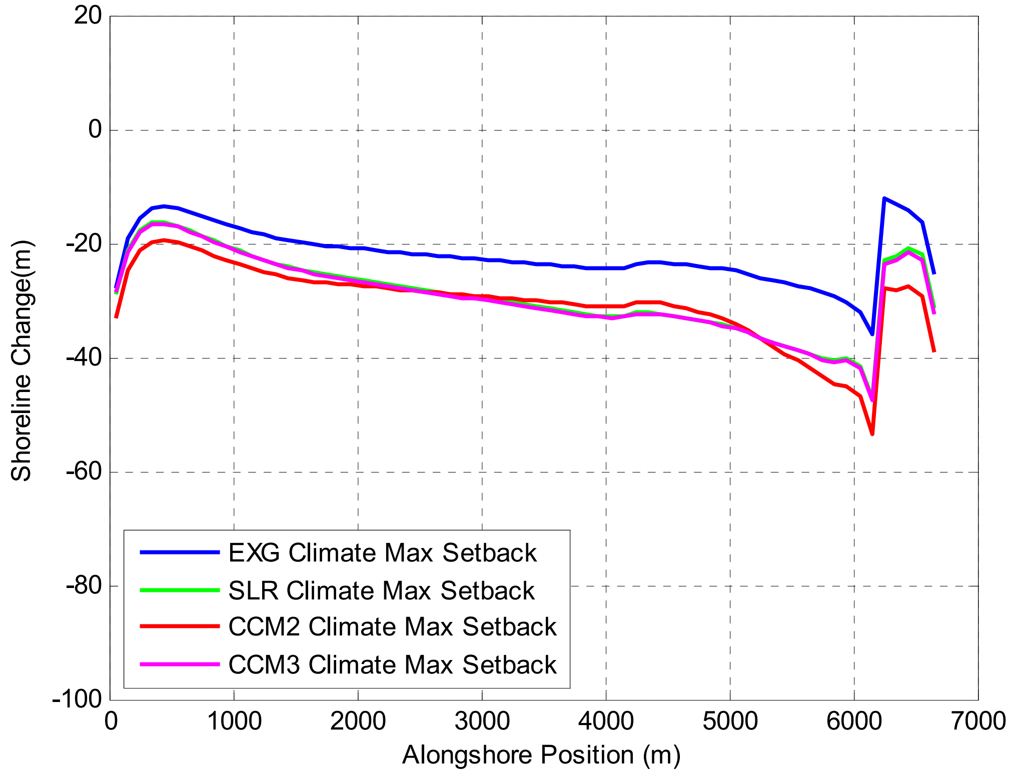


Figure 6-31 Wooli Wooli Shoreline Position Result Comparison: 2030 Planning Horizon

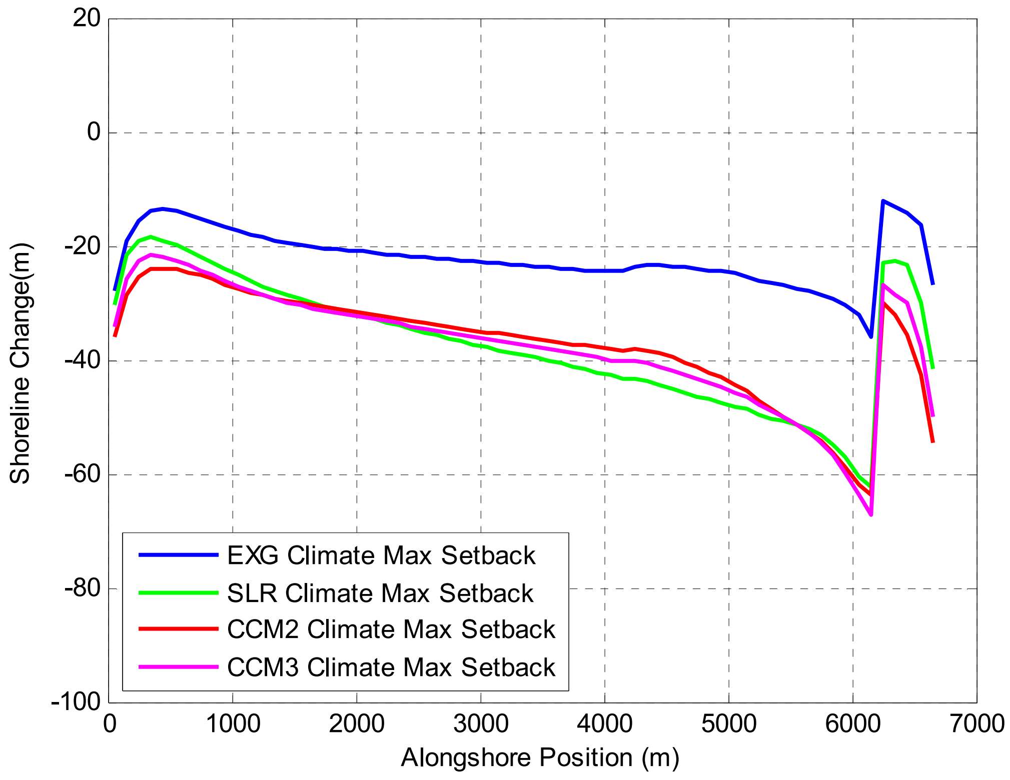


Figure 6-32 Wooli Wooli Shoreline Position Result Comparison: 2070 Planning Horizon

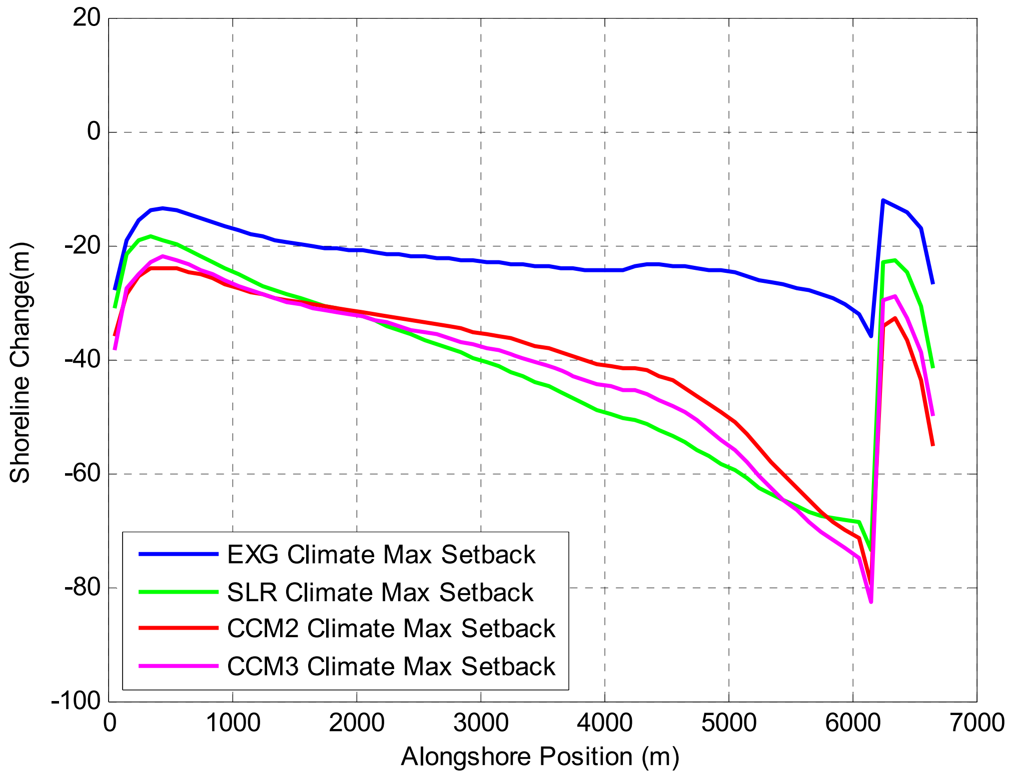


Figure 6-33 Wooli Wooli Shoreline Position Result Comparison: 2100 Planning Horizon

LEGEND

Initial Shoreline Position

MAXIMUM LANDWARD POSITION OF ALL CLIMATE CHANGE SCENARIOS

2030 Planning Horizon

2070 Planning Horizon

2100 Planning Horizon

Title:

Wooli Wooli Shoreline Position Results Summary

Figure: 6-36

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

N

0 1 2km

Scale - Map A

0 150 300m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : I:\B16444_I_BRH Climate Change IAT\DRG\COA_032_100506_WWSummary.wor

7 CONCLUSIONS AND RESULTS SUMMARY

Assessing the possible impacts of future climate change is a major issue currently requiring attention by governments worldwide. By assessing the likely impacts of climate change for the full range of environmental and regional sectors, governments will be able to prioritise adaptive strategies to manage the climate change impacts in an economically responsible way.

This study represents a portion of a NSW state government funded study assessing the possible impacts of climate change to the NSW coastline. As part of this study, a modelling approach capable of assessing the impact of climate change to the existing shoreline has been developed.

The developed modelling approach, based on existing coastal engineering theory, has been designed specifically to assess the impact of climate change on shoreline response for an open coast beach. The time stepping model further develops the Miller and Dean (2004) cross-shore model using a geometric representation of the cross-shore profile. The geometric approach has been applied using Bruun Rule style conservation of mass principles.

To assess the possible shoreline response to the combined impact of climate change driven sea level rise and variations in wave climate, developed cross-shore and longshore models have been dynamically linked. Combining of the modelling programs has enabled an efficient way to assess the likely impacts of climate change on shoreline response.

Specifically, the assessment of the likely impacts of climate change variations predicted by McInnes *et al.* (2007) on Wooli Wooli Beach has been undertaken using this developed modelling approach. A summary of the most extreme model scenario results for Wooli Wooli Beach is shown in Table 7-1.

Table 7-1 Wooli Wooli Climate Change Maximum Shoreline Recession Results

Result	*Scenario*	Planning Horizon		
		2030	*2070*	*2100*
Maximum Shoreline Change (MLSP)	EXG	-36.1	-36.1	-36.1
	SLR Only	-47.0	-62.2	-73.4
	CCM2	**-53.4**	-63.8	-79.5
	CCM3	-47.6	**-67.1**	**-82.5**
Maximum Climate Change Related Shoreline Recession (Climate Change Scenario minus Existing Case Scenario Result)	SLR Only	-10.9	-26.1	-37.3
	CCM2	**-17.3**	-27.7	-43.4
	CCM3	-11.5	**-31.0**	**-46.4**

Summarising the findings specific to Wooli Wooli Beach, using the developed assessment approach accounting for the combined impact of climate change on cross shore and longshore sediment transport processes, the results indicate there is a significant non-uniform alongshore response to the climate change forcing. The results show that greatest shoreline recession is likely to occur along the southern section of Wooli Wooli beach, adjacent to the northern training wall of the Wooli Wooli River. North of this location the shoreline recession results are reduced, with the northern section of Wooli Wooli beach exhibiting the smallest recession.

At its greatest, for the various assessed climate change scenarios, between 73m and 83m of shoreline recession is predicted at the southern end of Wooli Wooli beach, where the predicted erosion is greatest.

Comparing the various scenario results, it is evident that the projected increases in sea level dominate the shoreline response at Wooli Wooli. Projected changes in wave climate are predicted to have an effect of the future shoreline evolution, though the magnitude of change is comparably less than that resulting from sea level rise in isolation.

In the broader context of shoreline response to climate change, the Wooli Wooli assessment results highlight some interesting trends. The results indicate that shoreline response to sea level rise is highly non-uniform for littoral drift dominated coastlines. Where the annual net transport pattern dominates, such as at Wooli Wooli Beach, the modelling results indicate beach sections immediately downdrift from major headland/groyne controls are likely to experience the greatest shoreline recession.

These results highlighted the need to account for both cross shore and longshore processes during climate change assessments for littoral drift dominated coastlines. Traditional climate change assessments using the Bruun Rule, represent only a cross shore assessment, and do not account for the joint interaction of the shoreline evolution to combined cross shore/longshore processes. Using the developed time-stepping model approach outlined in this paper, more detailed shoreline response assessments are now able to be conducted, accounting for these complex sediment transport processes.

8 RECOMMENDATIONS

The developed dynamically linked cross-shore/longshore model represents an efficient way to assess the likely impacts of climate change on shoreline response. The modelling approach has enabled the assessment of the proportional impacts of various climate change induced changes to wave climate and water level for Wooli Wooli beach. As a preliminary assessment these results provide a significant insight into the shoreline response to variations in these forcings.

It is however recognised that additional assessments and further development of the XSMOD model may assist to create a more flexible modelling approach.

Possible updates to the developed modelling software include:

1. Inclusion of a profile input representing the maximum erosion depth (i.e. bedrock) in XSMOD. Currently the model assumes the is an infinite sediment supply from the dune to the depth of closure;

2. Coding of the XSMOD/LSMOD linking via an executable instead of relying on Matlab.

Additional research, which would assist to further validate the developed modelling approach include:

3. Research, identifying the parameters which define the offshore “transition” and “deep water” bed slopes.

4. Assessment of the developed modelling approach at additional case study locations using historic survey data. In particular, case study location exhibiting vastly different median grain sizes should be assessed. Via this assessment, the possible relationship between the XSMOD accretion and erosion rate constants, to sediment sizing could be identified.

In terms of the application of the modelling approach, the climate change assessment of Wooli Wooli Beach represents a preliminary assessment identifying the sensitivity of shoreline response to the predicted variation in wave climate and sea level.

5. It is recognised that the deterministic approach adopted during the climate change assessment for Wooli Wooli is less statistically robust than an approach based on a probabilistic framework (Callaghan *et al.* ,2008; De Vreind, 2003). Application of the developed modelling procedure using the probabilistic framework was outside to scope of the current assessment. The developed model is however well suited to the probabilistic application due to the computational efficiency of the approach. As such, it is recommended that further research using the developed modelling approach be undertaken using a probabilistic framework.

9 REFERENCES

Allen, M. and Callaghan, J. (1997) *"Extreme Wave Conditions for the South Queensland Coastal Region"*. Queensland environmental Protection Authority

Australian Government Department of Climate Change. (15/2/2008) *"Australian Government Action on Climate Change".* Australian Government Department of Climate Change, http://www.climatechange.gov.au/about/publications/pubs/fs-overview.pdf

Bindoff, N.L., J. Willebrand, V. Artale, A, Cazenave, J. Gregory, S. Gulev, K. Hanawa, C. Le Quéré, S. Levitus, Y. Nojiri, C.K. Shum, L.D. Talley and A. Unnikrishnan, (2007), *"Observations: Oceanic Climate Change and Sea Level. In: Climate Change 2007: The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change"* [Solomon, S., D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M. Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA

Bruun, P. (1954) *"Coast Erosion and the Development of Beach Profiles"*. Beach Errosion Board Technical Memorandum No. 44, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Bruun, P. (1962) "*Sea Level Rise as a Cause of Shoreline Erosion"*. Journal of Waterways and Harbours Division, American Society Civil Engineering, 88: 117-130.

Bruun, P. (1986) "*The Bruun Rule of Erosion by Sea-Level Rise: A discussion on Large Scale Two and Three-Dimensional Usages". Journal of Coastal Research, 4(4), 527-648. Charlotteville*

Bureau of Meteorology (updated 5/1/2009), *http://www.bom.gov.au/cgi-bin/climate/change/trendmaps.cgi*, Commonwealth of Australia 2009

Callaghan, D.P., Nielsen, P., Short, A., and Ranasinghe, R. (2008) *"Statistical simulation of wave climate and extreme beach erosion".* Coastal Engineering 55 (2008) pp375-390

Church, J.A., and N.J. White, (2006), "*A 20th century acceleration in global sea-level rise"*. Geophys. Res. Lett., 33, L01602, doi:10.1029/2005GL024826.

Cowell, P.J., Roy, P.S. and Jones, R.A. (1995) *"Simulation of Large-Scale Coastal Change Using a Morphological Behaviour Model"*. Marine Geology 126: 45-61

Cowell, P.J. and Thom, B.G. (1994) *"Morphodynamics of Coastal Evolution"*. In R W G Carter and C D Woodroffe (Eds) Coastal Evolution: Late Quaternary shoreline dynamics, Cambridge University Press, (pp 33-86)

Cowell, P. J., Thom, B.G., Jones, R.A., Evert, C.H. and Simanovic, D. (2006) *"Management of Uncertainty in Predicting Climate- Change Impacts on Beaches"*. Journal of Coastal Research, 22: 232-245

CSIRO website (2008) http://www.cmar.csiro.au/sealevel/sl_proj_regional.html, Last modified 18/11/08

Dean, R. G. (1977) *"Equilibrium Beach Profiles: US Atlantic and Gulf Coasts,"* Ocean Engineering Report No. 12, Department of Civil Engineering, University of Delaware, Newark, DE.

Dean, R., and Dalrymple, R. (2002) *"Coastal Processes with Engineering Applications".* Cambridge University Press.

Dean, R., (1987) *"Coastal Sediment Processes: Toward Engineering Solutions".* Proc. Coastal Sediments, ASCE, 1-24, 1987

Dean, R., Kriebel, D., Walton, T. (2002)"*Coastal Engineering Manual: Cross-Shore Sediment Transport Processes"*. Coastal and Hydraulics Laboratory – Engineering Research and Development Center Waterways Experiment Station – Vicksburg Mississippi

DECCW (2009*), "Sea level Rise Policy Statement"*, NSW Department of Environment and Climate Change, 2009.

Department of Environment, State and Territories. (1996) *"National Coastal Vulnerability Assessment"*. Case Studies Project, Commonwealth of Australia, Canberra.

Department of Environment and Climate Change. (30/1/2008) *"Department of Environment and Climate Change"*. NSW Department of Environment and Climate Change http://www.epa.nsw.gov.au/.

Department of Natural Resources. (2007) *"Consultant Brief for Stage 2 of DNR Climate Change Impacts Project: Quantitative Estimation of Coastal and Estuarine Responses to Climate Change".* NSW Department of Environment and Climate Change.

De Vreind. (2003) *"On the Prediction of Aggregated-Scale Coastal Evolution".* Journal of Coastal Research, 19(4), 757-759

Dubois, R.N. (1975) *"Support and Refinement of the Bruun Rule of Beach Erosion".* Journal of Geology, 83, 651-657

Dyke, Phil. (2007) *"Modeling Coastal and Offshore Processes"*. Imperial College Press.

Everts, C.H. (1985) *"Sea Level Rise Effects on Shoreline Position".* Journal of Waterways, Port, Coastal and Ocean Engineering, 111 (6), 985-999

Falque's, A. (2006) *"Wave Driven Alongshore Sediment Transport and Stability of the Dutch Coastline"*. Coastal Engineering 53 (2006) pp243-254

Forster, P., V. Ramaswamy, P. Artaxo, T. Berntsen, R. Betts, D.W. Fahey, J. Haywood, J. Lean, D.C. Lowe, G. Myhre, J. Nganga, R. Prinn, G. Raga, M. Schulz and R. Van Dorland (2007), "*Changes in Atmospheric Constituents and in Radiative Forcing. In: Climate Change 2007: The Physical Science Basis",* Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change [Solomon, S., D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M.Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA

Fowler, J.E., Rosati, J.D., Hamilton, D.G., and Smith, J.M. (1995) *"Development of a Large-Scale Laboratory Facility for Longshore Sediment Transport Research". The CERCular, CERC-95-2,* U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Gourlay, M.R, (1978) *"Wave Generated Currents"*. University of Queensland, PHD Thesis.

Greer, M.N., and Madsen, O.S. (1978) *“Longshore Sediment Transport Data: A Review”*. 16th Coastal Engineering Conference, ASCE, Hamburg, Chapter 93, Vol 2, pp.1563-1576.

Hands, E.B. (1983) *“The Great Lakes as a Test Model for Profile Responses to Sea Level Changes”*. In Komar, P.D., (ed.) Handbook of Coastal Processes and Erosion Boca Raton. CRC Press, pp176-189.

Hanson, H. and Kraus, N. (1989) “*GENESIS: Generalised Model for Simulating Shoreline Change: Report 1 Technical Reference”.* US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Hanson, H., Aarninkhof, S., Capobianco, M., Jime´nez, J.A., Larson, M., Nicholls, R.J, Plant, N.G., Southgate, H.N., Steetzel, , H.J., Stive, M.J.F. and De Vriend H.J. (2003) *“Modelling of Coastal Evolution on Yearly to Decadal Time Scales”.* Journal of Coastal Research, 19(4), 790-811.

Herbers, T.H.C., Orzech, M., Elgar, S., and Guza, R.T. (2003). *“Shoaling Transformation of Wave Frequency-directional Spectra”*, Journal of Geophysical Research, 108 (CI), No. 3013.

Hoffman,J.S, Keyes, D., and Titus, J.G. (1983) “*Projected Future Sea Level Rise: Methodology, Estimates to the Year 2100 and Research Needs”*. Washington, D.C: Government Printing Office.

Holgate, S.J., and P.L. Woodworth, (2004) “*Evidence for enhanced coastal sea level rise during the 1990s*.” Geophys. Res. Lett., 31, L07305, doi:10.1029/2004GL019626.

Huchzermeyer, E. (2005) *”Measuring Total Longshore Sediment Transport with a Lisst Instrumented Mini-Sled”.*Office of Graduate Studies of Texas A&M University

Hughes, S.A. and Chiu, T.Y. (1981) *“ Beach and Dune Erosion During Severe Storms.”* UFL/COEL-TR/043, Dep. Coastal Oceanograph. Eng., Univ. Florida, 1981.

Huxley, C., Teakle, I., Patterson, D., and Ranasinghe, R. (2007) *”Quantification of the Physical Impact on the NSW Coastal Zone Due to Climate Change”*. NSW Coastal Conference 2008, Yamba NSW.

Intergovernmental Panel on Climate Change. (2007) *”Summary for Policy Makers In: Climate Change 2007: The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change”* [Solomon, S., D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M.Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

Kampius, J.W. (1991) *”Alongshore Sediment Transport Rate”.* Journal of Waterway, Port, Coastal and Ocean Engineering, Vol 117, No. 6. pp624-640.

Kay, R. and Waterman, P. (1993) *“Review of the applicability of the common methodology for assessment of the vulnerability to sea level rise to the Australian coastal zone”.* In McClean,R., Mimura, N., (eds) Proceedings of the IPCC Eastern Hemisphere Workshop on Vulnerability Assessment to Sea Level Rise and Coastal Zone Management 237-248. Geneva: IPCC Secretariat World Meteorological Organization.

Kennedy, A.B., Chen, Q., Kirby, J.T., and Dalrymple, R.A. (2000) *”Boussinesq Modeling of Wave Transformation, Breaking, and Runup”*. Journal of Waterway Port Coastal and Ocean Engineering, 126 (91), 39-47.

King, D. (2006) *"Dependence of the CERC Formula K Coefficient on Grain Size".* Coastal Engineering 2006. pp 3381-3390

Kobayashi, N. (1999) *"Numerical Modelling of Wave Runup on Coastal Structures and Beaches"*. Marine Technology Society Journal, 33 (3), 33-37.

Kriebel, D. and Dean, R. (1985) *"Numerical Simulation of Time-Dependent Beach and Dune Erosion".* Coastal Engineering, 9 (1985) pp221-225

Kriebel, D. and Dean, R. (1993) *"Convolution Method for Time-Dependent Beach Profile Response".* Journal of Waterway, Port, Coastal and Ocean Engineering, 119 (2) pp204-226

Larson, M. (1989) *"SBEACH: Numerical Model for Simulating Storm-Induced Beach Change – Empirical Foundation and Model Development"*. Department of the Army Coastal Engineering Research Center Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

Leuliette, E.W., R.S. Nerem, and G.T. Mitchum, (2004) "*Calibration of TOPEX/Poseidon and Jason altimeter data to construct a continuous record of mean sea level change*." Mar. Geodesy, 27(1–2), 79–94.

Lin, P. and Liu, P.L.F. (1998) *"A Numerical Study of Breaking Waves in the Surf Zone"*, Journal of Fluid Mechanics, 359, 239-264.

Lockhart. J.H. and Morang, A. (2002) *"History of Coastal Engineering"*. Coastal and Hydraulics Laboratory (CHL), Engineer Research and Development Center, Vicksburg, Mississippi.

Lomborg, B. (2007) "*Cool It – The Sceptical Environmentalist's Guide to Global Warming"*, Alfred A. Knopf New York 2007.

Lynett, P., Wu, T.R. and Liu, P.L.F. (2002) *"Modelling Wave Runup with Depth Integrated Equations"*, Coastal Engineering, 46, 89-107.

Macadam, I., McInnes, K., O'Grady, J. (2007) *"Climate Change Projections for the Wooli Wooli Estuary and Batemans Bay".* Report for the New South Wales Department of Environment and Climate Change. September 2007

Maddux, B., Cowen, E., Foster, D., Haller, M. and Stanton, T. (2006) *"The Cross-Shore Sediment Transport Experiment (CROSSTEC)"*. Proceedings of Coastal Engineering: 30th Intl. Conf., ASCE, San Diego, CA, pp. 2547—2559, 2006

McInnes, K., Abbs, D., O'Farrell, S., Macadam, I,. O'Grady, J,. Ranasinghe, R. (2007) *"Projected Changes in Climatologically Forcing For Coastal Erosion in NSW ".* Report for the New South Wales Department of Environment and Climate Change. August 2007

Meehl, G.A., Stocker, T.F., Collins, W.D., Friedlingstein, P., Gaye, A.T., Gregory, J.M., Kitoh, A., Knutti, R., Murphy, J.M., Noda, A., Raper, S.C.B., Watterson, I.G., Weaver A.J., and Zhao Z.-C. (2007) *"Global Climate Projections. In: Climate Change 2007: The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change".* [Solomon, S.,D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M. Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

Miller, J. and Dean, R. (2004) *"A Simple New Shoreline Change Model"*. Coastal Engineering 51 (2004) pp531-556.

Miller, J.K. and Dean, R.G. (2006) *"An Engineering Scale Model for Predicting the Shoreline Response to Variations in Waves and Water Levels".* Proceedings of the 7th International Conference on Hydroscience Engineering Philadelpha USA 10th-13th September, 2006

Moore, B.D. (1982) *"Beach Profile Evolution in Response to Changes in Water Level and Wave Height".* MCE Thesis, Department of Civil Engineering, University of Delaware, pp164, 1982.

Patterson, D. (2007a) "Comparison of Recorded Brisbane and Byron Wave Climates and Implication for Calculation of Longshore Sand Transport in the Region". International Coastal Symposium 2007 Gold Coast.

Patterson, D. (2007b) *"Sand Transport and Shoreline Evolution, Northern Gold Coast, Australia"*. Journal of Coastal Research, Special Issue 50.

Pilkey, O.H., and Davis, T.W. (1987) *"An Analysis of Coastal Recession Models: North Carolina Coast"*. In: Sea-Level Fluctuation and Coastal Evolution: Society for Sedimentary Geology. Special Publication No. 41, Tulsa, Oklahoma, pp59-68

Port of Melbourne Authority. (1992) *"Victorian Coastal Vulnerability Study".* Coastal Investigations Unit, Port of Melbourne Authority

Pruzak. Z. (1993) *"The Analysis of Beach Profile Changes Using Dean's Method and Empirical Orthogonal Functions".* Coastal Engineering, 19, pp245-261, 1993

Rakha, K.A., Deigaard, R. and Broker, I. (1997). *"A Phase-Resolving Cross Shore Sediment Transport Model for Beach Profile Evolution"*, Coastal Engineering, 31,231-261.

Ranasinghe, R., McLoughlin, R., Short, A., Symonds, G. (2004a) *"The Southern Oscillation Index, Wave Climate, and Beach Rotation"*. Marine Geology 204 (2004) p273-287

Ranasinghe, R., Symonds, G., Black, K., and Holman, R., (2004b) *"Morphodynamics of intermediate beaches: a video imaging and numerical modelling study.* Coastal Engineering, 51 (7), pp629-655.

Ranasinghe, R., Watson, P., Lord, D., Hanslow, D., and Cowell, P. (2007) *"Sea Level Rise, Coastal Recession and the Bruun Rule".* Coasts and Ports Conference 2007 – Melbourne.

Rosati, J., Walton, T., Bodge, K. (2006) *"Coastal Engineering Manual: Longshore Sediment Transport"*. Coastal and Hydraulics Laboratory – Engineering Research and Development Center Waterways Experiment Station – Vicksburg Mississippi

Rosen, P. (1978) *"A Regional Test of the Bruun Rule on Shoreline Erosion"*. Marine Geology, 26, M7-M16.

Schwarz, M.L. (1965) *"Laboratory Study of Sea Level Rise as a Cause of Shoreline Erosion"*. Journal of Geology, 73, pp528-534

Schwarz, M.L. (1967) *"The Bruun Theory of Sea Level Rise as a Cause of Shoreline Erosion"*. Journal of Geology, 75, pp76-92

SCOR Working Group 89. (1991) *"The Response of Beaches to Sea Level Changes: A review of Predictive Models"*. Journal of Coastal Research, 7(3), pp895-921

Sharples, C. (2004) *"Indicative Mapping of Tasmania Coastal Vulnerability to Climate Change and Sea Level Rise".* Explanatory Report for the Department of Primary Industries, Water & Environment (Tasmania), 126 pp.

Shi, F., Svendsen, L.A., Kirby, J.T., and Smith, J.M., (2003) *"A Curvilinear Version of a Quasi-3D Nearshore Circulation Model"*, Coastal Engineering, 49, 99-124.

Slinn, D.N. and Riley, J.J. (1998). *"A Model for the Simulation of Turbulent Boundary Layers in an Incompressible Stratified Flow"*, Journal of Computational Physics, 144, 550-602.

Smith, E., Ping, W. and Zhang, J. (2003) *" Evaluation of the CERC Formula Using Large Scale Model Data".* Engineering Research and Development Center Waterways Experiment Station – Vicksburg Mississippi

Stive, M.J.F. (2004) *"How Important is Global Warming for Coastal Erosion?"*. Climate Change, Editorial Comment, 64, pp27-39

Titus, J., Leatherman, S., Everts, C., Kriebel, D. and Dean, R. (1985) *"Potential Impacts of Sea Level Rise on the Beach at Ocean City, Maryland".* United States Environmental Protection Authority, October 1985

Trenberth, K.E., P.D. Jones, P. Ambenje, R. Bojariu, D. Easterling, A. Klein Tank, D. Parker, F. Rahimzadeh, J.A. Renwick, M. Rusticucci, B. Soden and P. Zhai, 2007: Observations: Surface and Atmospheric Climate Change. In: Climate Change (2007) *"The Physical Science Basis.* Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change" [Solomon, S., D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M. Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

Van Dongeren, A.R., and Svendsen, L.A. (2000). *"Nonlinear and 3D effects in Leaky Infragravity Waves"*, Coastal Engineering, 41(4), 467-496.

Vellinga, P. (1982) *"Beach and Dune Erosion During Storm Surges".* Delft Hydraulic Laboratory

Vellinga, P. (1983) *"Predictive Computational Model for Beach and Dune Erosion During Storm Surges".* Delft Hydraulic Laboratory

Voice, M., Harvey, N. and Walsh, K. (2006)*"Vulnerability to Climate Change of Australia's Coastal Zone: Analysis of Gaps in Methods, Data and System Thresholds".* Report to the Australian Greenhouse Office, Canberra, Australia. June 2006

Wang, P., Kraus, N.C., and Davis, R.A. (1998) *"Total Rate of Longshore Sediment Transport in the Surf Zone: Field Measurements and Empirical Predictions"*. Journal of Coastal Research, 14(1), 269-283.

Zedler, E.A. and Street, R.L., (2002). *"Nearshore Sediment Transport: Unearthed by Large Eddy Simulation"*, Proc. 28th International Conference on Coastal Engineering, ASCE, 2504-2516.

Zhang, K., Douglas, B.C. and Leatherman, S.P. (2004) *"Global Warming and Coastal Erosion"*. Climate Change, 64, pp41-58.